Structural Engineering in Nigeria *

by

A. Brimer, A.M.I.Struct.E.

Synopsis

The paper deals with the general geographical and economic characteristics of Nigeria and proceeds to show how the profession of structural engineering is represented in that country. The work of the engineer in the office and in the field is then described and some of the commoner engineering problems encountered in Nigeria are illustrated. The paper concludes with a brief reference to the future of the country and the role of the engineer in its development.

The Country

Sixty years ago the Colonial Correspondent of *The Times*, Flora Shaw, suggested the name Nigeria for the territories around the Niger over which Britain had established a protectorate. In 1900 Britain took over control of the Colony of Lagos, the Niger Coast Protectorate, the Royal Niger Company Protectorate, which in theory extended as far as Katsina and Bornu, and called the new territory Nigeria, which was then divided into North and South. Today, sixty years later, that territory, carved arbitrarily out of the coast of Africa, becomes a sovereign independent state, and by virtue of its size and population will yield an immense influence not only in Africa but throughout the world.

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Today the country is divided into three main Regions, the North, West and East. Each Region has its own Government and also elects its own members for the Federal Government which sits in Lagos.

The Federal capital of Lagos occupies about twenty-seven square miles and reclamation of swamp land, demolition of slum areas, extensive rehousing estates and the construction of buildings are transforming it into a lively modern city, teeming with the life of its 350,000 citizens. Lagos is a cosmopolitan city where Nigerians from every part of this vast country meet with visitors from East and West, represented by officials, traders, journalists, students, businessmen and visiting diplomats and politicians.

The Northern Region of Nigeria covers more than three-quarters of the total area of the Federation (281,788 square miles) and has half the total population (over 18 millions). Best known of the Northern cities is Kano, for more than 1,000 years one of the great central marts of the Sudan and now a meeting place of the new type of caravan that comes in from the sky. Kaduna, capital of the North, was originally intended to be the capital of Nigeria. In the far North-West is Sokoto whose ruler, the Sultan of Sokoto, is also the Sarkin Musulmi or leader of all Muslims in Western Sudan.

Northern Nigeria was the last of the regions to become self-governing but rapid strides are being taken to hasten the economic, social and industrial development of the people of the North.

The Eastern Region of Nigeria is the principal home of the Ibo people covering some 29,484 square miles, and is the most densely populated with a population of about eight millions. Enugu is the capital of the Region and is built near the hills through which run the coal seams that are Nigeria's principal domestic source of fuel for industrial undertakings. Within the past few years mineral oil has been found in commercial quantities in the swampy delta area and by the end of 1960 output was expected to reach a rate of 1½ million tons per year.

The Western Region of Nigeria covers an area of 45,376 square miles, and its population of 6,352,472 is the smallest of the three Regions in the Federation. There are highly organised indigenous political and social systems and the Region is noted for the large number of excellent works of art. The best known Yoruba arts are the Ife bronzes and terra-cottas, also the famous Benin bronzes and wood carvings. Ibadan is the capital town of the Western Region and has a fine new Parliament Building. Adding beauty and importance to Ibadan are the Federal Institution of the University College and the University College Teaching Hospital. The Region is rich in agricultural products such as cocoa.

The natural vegetation of Nigeria consists of a belt of high forest, generally about 100 miles wide, which stretches right across the Southern part of the country. It is in this belt of high forest that the valuable timber trees are found. North of this belt lie the savannah areas which extend with diminishing intensity, as far as tree growth is concerned, up to the northern boundary of Nigeria. They are subdivided, going from South to North, into Southern and Northern Guinea Zone, both consisting of fairly good savannah woodland, the Sudan Zone which is drier type with less grass and finally, over a small area, the Sahel Zone consisting of open savannah and thorn trees. The average annual rainfall varies from 40 to 80 inches per year in coastal areas to 20 to 40 inches in the North. Daily temperatures average a maximum of 85°F on the coast, and 98°F in Northern areas. The relative humidity, which is an index of the 'comfort' of the human-being, is as high as 85 per cent along the coast, but drops to 55 per cent in the savannah area.

Communication

Nigeria has had some form of commerce with the outside world for many years but the end of the slave trade in 1840 saw the advent of legitimate maritime trade which was then carried on by sailing vessels. Up till 1959, the export and import trade of Nigeria was carried out by ships owned and operated by foreign firms. As a result of a growing consciousness among Nigerians of the need for a merchant fleet controlled and operated by Nigeria the National Line was incorporated in 1959.

Changes have taken place over the last fifty years—in 1912 the total tonnage of imports and exports was 380,000, today it represents some four million tons.

To improve the ports facilities the Nigerian Ports Authority was established in 1955 with the sole purpose of organising port management and harbour administration. Since then the turn-round of ships has greatly improved. Port facilities, such as deeper channels, have been considerably increased and in the case of Lagos, navigation during the hours of darkness has been introduced. The Authority's quays in Lagos and Port Harcourt handled a total of 2,080 ships in the year 1958-59, an increase of 706 ships over the four years since the formation of the Authority.

The port of Lagos when taken over by the Ports Authority consisted of four berths and some transit sheds. With the completion of the Apapa Wharf extension in 1956, four new berths were added together with more transit sheds and ample space in the harbour area for the stacking of heavy cargo. The total length of the quay is 4,525 feet, which will accommodate eight or nine vessels.

Port Harcourt, which is 41 miles from the sea and is reached through the Bonny River, was originally planned as a port terminal for the eastern branch of the railway, but the rapid expansion of Nigeria's overseas trade since the end of the second world war has placed a severe strain on existing port facilities and a new extension, providing three new berths together with new transit sheds, offices, road and rail approaches and a modern marshalling yard, is under construction.

Port Harcourt handles traffic from the Eastern Region and North Eastern portion of the Northern Region as well as transit traffic to and from the French Chad.

The railway was built in Nigeria before the roads, except those roads within the limits of townships; it has therefore played a vital part in the progress and development of the country. In 1901 the railway brought the Lagos Government into close contact with the towns of Abeokuta and Ibadan.

Freight traffic has increased almost five-fold within twenty-five years; from 1933-34, 627,000 tons to 1958-59, 3,097,000 tons. Groundnuts are railed from the Northern Region, Kano area; cotton from Funtua and Gusua area and other principal products including coal, tin, columbite, palm produce, hides and skins are railed to the ports.

A wide range of development works costing over £40 million is in progress to relay tracks, remodel stations and purchase of new rolling stock. But the most important project is the extension of the railway for four hundred miles from Kuru on the Bauchi plateau, to Maiduguri in North-East Nigeria.

The aeronautical facilities necessary for international and internal air services serving Nigeria today are of a relatively complex nature. In the early days no special facilities were provided but the growth of aviation has since culminated in the need for expensive equipment and a network of specially designed airports and aerodromes.

In 1935 plans were made to start airmail services between the United Kingdom and Nigeria, via Khartoum to Kano and thence to Lagos through a chain of small grass and laterite landing grounds. After the war international services were introduced and the distances the aeroplanes could cover without refuelling had increased by this time, and instead of following the route down the Nile to Khartoum and then to Nigeria, they took the much shorter route from Europe to Tripoli and across the Sahara to Kano.

In 1946 the West African Airways Corporation was formed, which was subsidised jointly by Nigeria, the Gold Coast (Ghana), Sierra Leone and the Gambia, with its administrative headquarters situated at Ikeja, the Lagos Airport. Altogether, there are two major airports and twenty-six aerodromes in Nigeria. Kano is the major airport with a runway of 8,600 feet, and the average yearly number of aircraft movements is about 8,800.

Lagos Airport is mainly a terminal airport which serves the south-end area of Nigeria, and the main runway has now been extended to 7,600 feet.

The first roads for motor traffic were built in about 1900 and were feeder roads to the newly constructed railway line from Lagos to Oshogbo. At that time

there were no roads running northwards from Lagos. From 1900 to 1910 there was considerable railway construction and care was taken that roads did not compete with the railway. By 1927 a trunk road policy was formulated and roads ceased to be constructed purely as feeders to the railway system. Trunk roads 'A' which are constructed and maintained from Federal funds, are in the form of a rigid framework on which the rest of the road system is built up. Trunk roads 'B' connect provincial or divisional headquarters and other large towns with the trunk 'A' system, with one another and with points on the railway. They are controlled by the Regional Governments and maintained by native authorities or local government bodies.

In 1945 the first road financed by the Colonial Development and Welfare fund was begun, across the very difficult swamps from Lagos to Ikorodu where it joined the road to Shagamu and Ibadan. Road construction undertaken at that time was based on an estimated requirement of 40,000 miles of road. The total mileage of roads in Nigeria and the Southern Cameroons was 24,600 out of which only 600 miles had been tarred. By 1960 the road mileage had increased to approximately 6,000 miles of trunk 'A', about the same mileage of trunk 'B' and approximately 160,000 miles of local roads. Many of the local roads, particularly in the Eastern Region, were constructed by voluntary community labour.

Steady progress has been made with bridges. Most of the earlier timber bridges have been replaced by modern ones designed to current United Kingdom standard strengths. Some bridges were for reasons of economy built to take only one lane of traffic, but many of these have since been widened to take two lanes. Two bridges recently completed in Lagos have been built of prestressed concrete. Many of the larger bridges are of steel construction, the most notable examples being the Jebba bridge over the Niger and the bridge over the Benue river at Makurdi, both being road and rail bridges, and Carter Bridge in Lagos which is over 2,000 feet long and is the only outlet from Lagos island to the mainland.

The relief map of Nigeria shows that the rivers and creeks of this vast country stand out prominently as one of its important economic features. They provide not only a means of transport and communications to places which are inaccessible by road or rail, but some of them, if used effectively, are the sources from which hydro-electric power can be produced and from which, through irrigation, large areas of dry land can be made productive. Approximately 4,000 miles of these waterways are navigable by river craft, and during the high river season one can travel by water on the River Niger as far north as Jebba and Yola on the River Benue.

The Electricity Corporation of Nigeria was established in 1950 and was faced with a large potential demand for electrical energy, with plant and equipment which were not only in an extremely poor state but the capacity of which was quite inadequate to meet this potential demand. In 1951 units generated were approximately 61,000,000 Kw.H. and the estimated figure for 1960 was 359,000,000 Kw.H.

A very important investigation of the hydro-potential has been in progress for some time in the area of Jebba on the river Niger, and a smaller scheme at Shiroro Gorge on the Kaduna River. The two schemes combined are based on a capacity of over 2,000 M.w, at an approximate cost of £163 millions.

The initial development consists of a dam at Kurwasa which is estimated to cost £57 millions and will not only provide electricity but will bring the whole river under control, thereby making it navigable up to Kiarney in the neighbouring French territory.

The average consumption of electricity per inhabitant has doubled since 1951 but even this represents only 8 Kw.H. per head per annum.

The Profession in Nigeria

The title of this paper, "Structural Engineering in Nigeria" naturally implies the existence of our profession in that country, and members may be interested to know something of the representation of the Institution in Nigeria. Reference to the current edition of the Year Book shows that there are 56 members of all grades practising in various parts of the Federation. As is to be expected, the majority of them are concentrated in the larger towns, Lagos having 26, half of the total membership. Apart from private practice, these men are to be found in every branch of the engineering world, the greatest number of them being employed by various Government departments. Others are engaged by the Railways, Ports Authority, Local Government and private enterprise, and the work they do is as varied and interesting as that in any other country.

Rather more than one-third of the total membership is composed of Nigerians, a fact that is worthy of note. Although the proportion of Nigerian engineers is very small in relation to the population of the country, their numbers are growing, and since the great need of emergent Africa is for trained men, it is in the highest degree important that such growth should be fostered and maintained. The road to qualification is not an easy one for the Nigerian wishing to enter the profession, for technical education, is, as yet, limited in both quantity and quality; the barrier of language, and especially the technical language of the engineer, has to be overcome, and financial stringency often prevents study in overseas countries. Despite these difficulties Nigerians are becoming structural engineers and bringing with them the sense of responsibility and integrity so necessary to the upholding of the dignity of our profession.

Those of you who assiduously study all that the Institution's Journal contains will be aware that a Nigerian Section of the Institution has been recently formed. That such a Section has been established is no small achievement, and, although it is still young, we in Nigeria have great hopes of it becoming not only a source of encouragement to aspiring Nigerian engineers but also a means of sharing and disseminating our knowledge of the special engineering and ethical problems encountered in Nigeria.

As far as relationships with other professions are concerned, such as architects and quantity surveyors, the overseas engineer is on the same footing as his colleagues in Britain. Each profession maintains its own ethical code, and usually has its own Branch or Association. At present, contact between these bodies is noticeably limited but this will doubtless change as membership grows and travelling facilities improve and it should not be forgotten that the climate, particularly in the coastal zone with its humid heat, is still capable of engendering that sense of listlessness which is common to tropical countries.

Engineering in the Office

Novelists as a rule do not seem to devote a great deal of their writings to the engineer as a fictional character. There is, no doubt, some very good reason as there must be for introducing him as a very stereo-

typed sort of fellow who, curiously enough, has nearly always spent years in some foreign country, usually in the tropics. Perhaps it is this partiality for hot climates that invariably leads to his portrayal as a bronzed and rugged outdoor man, for he is seldom associated with an office to which he can retire even if it is only to escape the heat. One would deny the existence of such engineers with caution but the fact remains that work in the office is as much a part of the engineer's life in tropical Africa as it is here in Britain.

The primary role of the engineer in the office is the preparation of the design, drawings and specification for the structure he is going to build and in these endeavours he is subject to regulations which govern what he can and cannot do. This is no less true of Nigeria than it is of Britain, and, being a country which has had many years of British rule, it is only to be expected that Nigeria should largely adopt British methods of engineering insofar as local administration and design and construction control are concerned. How far these methods are applied or enforced depends mainly upon the degree of urbanisation of the area where construction is to take place ; in remote areas the engineer is pretty well left to his own devices but in the more populated centres he will find much that is familiar in British practice.

Major towns usually have a Town Engineers' department which operates in a manner similar to that of comparable organisations over here. Where a local authority does not include this department use is frequently made of the local office of the Public Works department. Few of these departments issue their own bye-laws, and engineering work in these localities is often based on the standards required by the major towns or the relevant British Codes of Practice adapted to suit local conditions. Not all centres, for example, have the facilities for dealing with calculations or approving drawings before construction is begun. Those of you who view this lack of formality with envy should not forget that such freedom is not without its disadvantages ; it very often means that there is no central pool of information upon which to draw, and a good deal of travelling and investigation may have to be done before deciding upon the type and method of construction to be used. The Federal Government has incorporated certain building laws in its statutory legislation, and due observance of these is obligatory.

Government departments, statutory bodies, and some larger private organisations will often issue their own recommendations and specifications, or require the engineer to specify the design stresses and loadings he intends to utilise and, in the latter case, considerable judgment and local knowledge is necessary, covering not only foundation conditions but also the availability and type of materials, competence of local contractors, and the supply of labour. This will be dealt with more fully in a later section.

So far the emphasis has been on the problems associated with design and construction in the more out-of-the-way places. We will now consider the regulations which exist in the larger towns. Lagos, because it is the capital, and most sophisticated town of the Federation, will be taken as a general example.

Lagos proper is situated on an island and the immediate hinterland is known as the Colony of Lagos, although both areas are referred to in general as 'Lagos.' Apart from the Lagos Town Engineers' department, which also controls construction in the Colony area, there is a planning authority, the Lagos Executive Development Board, whose duties parallel the Town Planning departments extant in Britain.

Without going into detail, the bye-laws issued by the Town Engineers' department follow broadly their British counterparts in relation to general building construction, with suitable modification to allow for local variation in materials used and the special needs of construction in tropical climates. Those sections dealing with structures in steel and concrete specify that the requirements of BSS 449 and CP 114 and Chap. V loading shall be met. There are, however, restrictions placed on the use of CP 114 in reinforced concrete design, and it is worth examining these in more detail.

The major differences lie in the allowable stresses which are permitted in concrete and the bye-laws specify two distinct types of structural sufficiency in this connection.

1. Ordinary structures. These are denoted by the use of concrete in which the design stress in bending does not exceed 600 lb/sq. in.
2. Special structures. Such structures are those in which the concrete bending stress exceeds 601 lb/sq. in. Two subdivisions are stipulated ;

2(a) Concrete bending stress between 601 and 750 lb/sq. in. In this case a part-time resident engineer or full-time Clerk of Works must be employed on the site. 'Part-time' is customarily taken to mean 'frequent site inspections by the supervising engineer.'

2(b) Concrete bending stress between 751 and 1,000 lb/sq. in. Here the requirement is that a full-time resident engineer is employed, who must be wholly engaged in superintending that site only.

Apart from the above restrictions, special structures can only be approved if the following conditions are complied with :

(a) a special form indicating, inter alia, the qualifications of the resident engineer, and the design stresses, must be lodged with the Town Engineers' Department, and approved by a special Structures Committee convened by the Town Engineer ;

(b) only contractors who have been approved for the construction of such buildings are employed.

One peculiar point about the specified concrete stresses is that no account is taken of the higher permissible stresses where richer mixes are concerned. In consequence it is standard practice to specify a nominal 1 : 2 : 4 mix, whatever the design stress used, and to rely on the supervision provided and the use of test-cubes to ensure that the required strengths are obtained.

Proposals for the erection of a structure of any type must be accompanied by the necessary design calculations, full working drawings, reinforcement details, and architectural drawings where applicable, and ordinarily approval is withheld unless complete sets of drawings are deposited. Although this obligation could lead to delays in the commencement of construction, particularly in the case of large structures, consideration is usually given where this proviso is likely to prove onerous.

Engineering in the Field

We now turn to that aspect of structural engineering which is concerned with actual construction. This is where the novelists' engineer comes into his own and without further ado we will place him on the open site, face to face with the contractor !

Basically the only difference between contractors in Britain and those in Nigeria is that in Nigeria they come in a much wider range of nationalities! A quick look around the various sites in Lagos, for instance, will show Italian, German, Dutch, French, Nigerian, Syrian and British firms. Their internal organisations may differ in keeping with the practice of their own countries but on the site they are very much the same and each contributes to the continual changes in the face of Nigeria. Many of them have had to assimilate the special peculiarities of British contracting practice, and this they have done with alacrity, with the result that today competitive tendering is remarkably keen.

The visitor from Britain would find several points of difference, however, in the constructional methods used in the two countries; steel scaffolding is rarely seen and the various arrangements of timber poles often surrounding the taller buildings would cause consternation among those concerned with the enforcement of the British Factory Acts; formwork for reinforced concrete is, again, completely fabricated from timber and although the use of steel shuttering has begun to make some headway the lack of hire facilities has militated against its general use; mechanization is not yet a dominant force in construction techniques, mainly on account of the large supply of unskilled labour; there is no ready-mixed concrete, and concrete-mixers larger than the 7T are exceptional; reinforcement is invariably cut and bent in situ, for the specialist supplier and contractor has not yet appeared on the Nigerian scene.

Concrete is conveyed from mixer to point of placing by means of the 'headpan,' a shallow circular metal basin some two feet in diameter which holds about 1½ cubic feet of material and is, as name implies, carried on the head. The headpan has many uses in addition to transporting concrete; removing spoil from excavations, carrying aggregates and mortar, and, at the end of the day's work, as a convenient bath. Probably nothing is more evocative of West African building than the sight of a line of labourers, each with his carefully balanced headpan, moving about the site.

In case these illustrations should convey the impression that construction is not far removed from the primitive, it must also be said that the tower crane is becoming a familiar sight on the Lagos skyline and few buildings are erected without the use of the materials hoist, power-tools of various descriptions and similar modern aids to speed and economy.

Practically the whole of the industry's labour force is Nigerian, with the exception of supervisory grades, and in the latter connection many firms have training schemes which have the eventual aim of producing site and office staffs which will be entirely Nigerian.

In common with many undeveloped countries, Nigeria is deficient in many ways of the constructional materials commonly used in Britain. The main reason for this is, of course, that it *is* an under-developed country and as progress in industrialization is made so will the situation improve. Already cement is being produced and plans are in being to provide steel-rolling mills and similar developments.

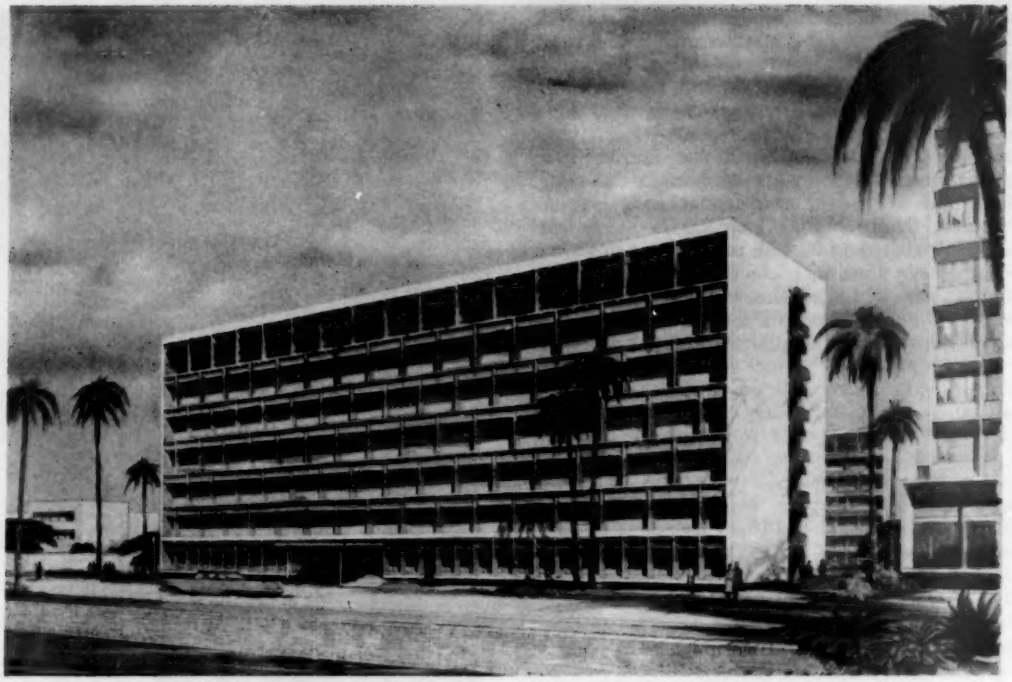
A brief survey of the more important materials of structural engineering commonly used in Nigeria will now be made.

As mentioned above, cement is being produced at two works at present. Quality is equal to the demands of BSS 12 but no special types, such as rapid-hardening or sulphate-resistant cements, are available from local sources. Imported cement is usually from British and Continental suppliers and is shipped in standard bags or in steel drums. As a point of interest, spoilage of imported cement due to humidity has been largely eradicated since the introduction of polythene bag linings.

Typical 16 storey office building under construction today in Lagos.

Steel sections and reinforcing rods are entirely imported from a number of different countries. Britain and the Benelux area form the largest suppliers, but occasional imports have been made from Japan and South Africa among others. Quality tends to be variable and care has to be exercised to ensure that steel, especially reinforcement, is capable of meeting the requirements of the relevant BSS. Reinforcing bars are commonly shipped in sizes from ¼ in. to 1 in. diameter, in increments of ⅛ in., the maximum length being about 30 ft. Any specific diameter or type of reinforcement can be obtained to order ex overseas stockists and it is expected that locally twisted (cold-worked) bars will soon be available. A growing amount of structural steelwork is being fabricated from locally held stocks but a considerable proportion is shipped from the United Kingdom.

Concrete aggregates are of local origin and vary widely in quality. The raw material is comparatively scarce and often results in long haulage problems. Coarse aggregate is produced by crushing and screening and although invariably specified to comply with BSS 882, it is doubtful if the majority of it would meet with all the requirements of that specification.



Seven-storey building due to start construction this year in Lagos.

One rather unusual means of producing coarse aggregate in the far north is worth mentioning; specially manufactured sand/cement blocks, crushed as necessary. Sand is dredged, more often than not by a skin diver with the ubiquitous headpan, or suction-pumped from river-bed deposits, and also shows a fairly wide range of quality.

Timber is one of Nigeria's exports although comparatively little use of it is made as a purely engineering material mainly because of variable climatic conditions, in particular the harmattan, a cold dry wind from the Sahara, which causes substantial warping and checking. It is, however, used widely for formwork and joinery. Most locally produced timbers are hardwoods, such as mahogany, and scantlings are seldom over twelve feet long due to transport difficulties. A high quality plywood is produced in Nigeria and this is used extensively in furniture production.

Bricks have a very limited use as a constructional medium, those which are made locally being of indifferent quality, soft and very susceptible to weathering unless protected by mud or cement rendering. Since there are deposits of both suitable clays and coal it is possible that a brickmaking industry will be established in the future. In place of bricks use is made of the conventional concrete block, solid or hollow, and these are generally manufactured on the site with the use of an ordinary hand press.

Virtually all other constructional materials are imported although it is advisable to say that this is the position at the time of writing. Rapid strides are being made to remedy this state of affairs and it is expected, for example, that a good deal of the country's requirements of paint, asbestos-cement and aluminium -heeting will shortly be produced in Nigeria.

Engineering Problems

The combination of the points raised in this paper so far all contribute to the existence of various problems which, although not unknown in Britain, tend to be accentuated in Nigeria. Much more care has to be taken at the design and detail stage to ensure that the structural requirements can be met with the materials available, and the engineer who specifies materials, which may be available in coastal areas, for use in construction up-country, may find himself faced with severe transport and economic difficulties. In the same way it is no use demanding specialised construction techniques in areas where they cannot be carried out. Paramount, therefore, in the thoughts of the good designer are three questions: can I get it? can it be done? what will it cost? With humility it is suggested that those engineers practising in Britain who are commissioned to design and draw up specifications for structures overseas should also bear these questions in mind and reduce their demands accordingly. This should not be construed as an invitation to lower standards in general but as a plea to relate such standards to the ability of the country to satisfy them. Two instances can be quoted of requirements which were almost impossible to fulfil, both of which were contained in specifications produced in England. In the one the use of imported British cement was laid down, it being clearly stated that such cement was not to be more than two months old. This is only very slightly longer than the shipping time between the manufacturer and Lagos! In the other, the specification for coarse aggregate necessitated the use of different clear sizes, to be re-combined on site to a special grading, an estimable adjunct to the production of good concrete which, however, is not yet readily available in Nigeria. Engineers in Nigeria are fully

aware of the need for the incorporation of modern techniques and the construction industry is itself making great efforts to promote their adoption.

The majority of our engineering problems in Nigeria are the same as those facing engineers the world over : founding multi-storey buildings in the mud of Lagos Island with a water table two feet down ; piles that seem to go on and on ; structural settlements on reclaimed ground ; these are common problems. Less familiar to engineers are some of the following : the predicament of the contractor whose carefully fixed slab and beam reinforcement vanished overnight ; the work-party which, with a completely clear area in which to fell a tall tree, managed to place it with uncanny accuracy across the centre of a new steel bridge with grievous results ; what to do when column starter bars have been neatly sawn off flush with the top of the footing and welding has been expressly prohibited ; and lastly, how to propitiate both gods and local dignitaries after unwittingly chopping down their ju-ju tree to make way for a new school.

Conclusion—Future

Whether or not one agrees with the materialistic outlook, there can be no doubt that the desire for material gain is now more evident in mankind than it has ever been. The demand for increased standards of living and prosperity is not confined to those lands with a long history of civilization but is now extant in practically every part of the globe. To satisfy those aspirations means that a great deal of development work is necessary and it is here that the engineer plays a predominant part.

Nigeria, in common with the majority of countries which have been in contact with the more highly developed nations for a comparatively short time, is subject to pressures for the provision of many of the things which we take for granted in addition to those for which we are still striving. In consequence, the problem which exists is not what to do but what to do first with the money which is available and it is not always easy to cajole an ingenuous people into understanding that an extra mile of road or a new dam is more important to the well-being of the country as a whole than is the acquisition of personal property.

Prophecy being largely the prerogative of wise men or fools, and engineers not being particularly noted for their inclusion in either group, it is perhaps injudicious to attempt to predict the road along which Nigeria will travel in the future. However, present indications are that the expansion of every branch of the Federation's economy is assured and that we, as engineers, will be called upon to play an ever-increasing part in the building of this great country.

Acknowledgement

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Book Reviews

Plastic Design in Steel. (Published by the American Institute of Steel Construction, New York, U.S.A.,1959) 12 in. × 8½ in. 53 + 41 pp. Appendices, 32s.

The American Institute of Steel Construction has adopted Rules for Plastic Design in Steel to supplement its well known Steel Construction Manual and Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings. These Rules are contained in a new Manual, *Plastic Design in Steel*, which reviews the basic concepts underlying the plastic theory and illustrates the use of the Rules by means of practical examples.

The book has been written to assist the practising designer ; it contains much tabular material and numerous charts compiled with the object of reducing design time to the minimum. It is not a complete text book, although it deals with most of the theory underlying plastic design. The book answers in a practical way many questions to which authoritative answers have not before appeared.

The book opens with a brief statement on the plastic theory and then proceeds to the analysis of single storey single and multiplebay frames, first by the graphical method and then by the mechanism method. Solution is aided by reference to 28 specially compiled charts. The effect of axial load on bending resistance is discussed for columns bent in single and double curvature and rules are given for design. A formula for bending resistance in webs at joints is given together with rules for the design of tapered and curved haunches : a noteworthy new feature is a rule for lateral bracing requirements. About half the book is devoted to practical designs of continuous beams, single span and multispan single storey frames : these designs are complete to the last detail and are most useful.

This publication can be recommended to all with the utmost confidence and is a noteworthy contribution to our engineering literature. Students will find it easy to read and assimilate, whilst designers will find it a great time saver, and those concerned with drafting specifications could read it with advantage.

C.S.G.

Ordinary Differential Equations and their Solutions, by G. M. Murphy. (New York & London : D. van Nostrand Co., 1960). 9 in. × 6 in., 451 + ix pp. 64s

This is a comprehensive reference book arranged for the most convenient use by engineers, those engaged in applied science and students.

The introductory chapter explains the arrangement, gives some definitions, some general properties of differential equations and a description of the symbols used in the book. This is followed by Part I which contains most of the methods which have been developed for solving ordinary differential equations. By following the directions the given equation can be classified as a special case of one or more general types. When the type has been identified, details of the method of solution can be found. Part II contains more than two thousand ordinary differential equations, arranged in systematic order. A solution of each is given or some directions as to a method. References to the general methods of Part I are included in each case.

A seven page bibliography, a general index and an index of equation types complete this useful volume.

Composite Construction in Theory and Practice*

by

Professor Dr. tech.h.c. Dr.-Ing. Konrad Sattler, Berlin

1. Introduction

In this paper, it will be assumed that composite construction is achieved when the complete cross-section of a concrete component acts with any steel section to form an integral unit. The term can be applied both to structures comprising steel beams and to prestressed concrete construction, but it does not apply to conventional reinforced concrete, where large zones of the concrete are considered as having cracked and become ineffective under working conditions.

Although the primary purpose of this paper is to describe composite construction embracing steel beams, both with and without prestressing, mention will be made when the bases of calculation can also be applied to prestressed concrete construction.

Composite construction gives a designer enormous scope. The concrete slab can be constructed in-situ or from precast elements. The steel beam can be a rolled or built-up section. It can have a solid web or it may be a lattice girder. The shear connection between the steel and concrete can be rigidly or elastically attached. The advantage of composite action lies in the fact that the slab not only spans and distributes the load over the main girders but also forms a part of them. It is possible to make large reductions in steel consumption with the most simple construction. Already in Germany, in the face of unceasing competition from prestressed concrete, a large number of highway bridges with free spans of from 50 to 100 metres have been erected

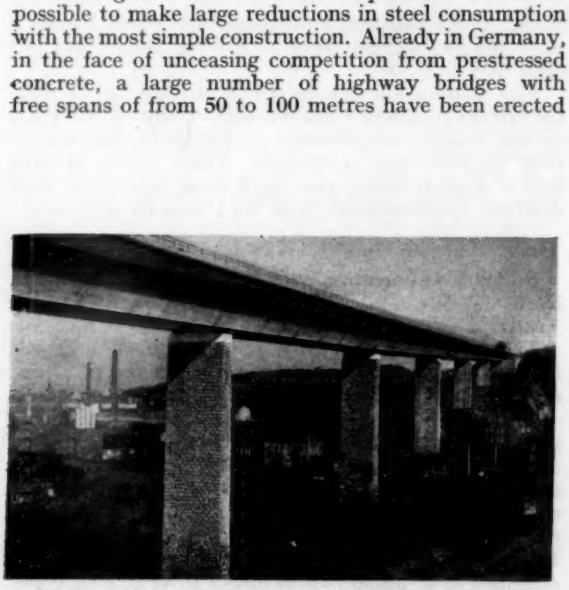


Fig. 1.—The Wuppertal-Oehde bridge.

Fig. 2.—Thyssen Hochhaus, Düsseldorf.

in composite construction, a recent example being shown in Fig. 1. Composite beams have also been used with advantage in office blocks, as shown in Fig. 2, and especially in industrial buildings.

A not unimportant requirement for the use of a particular type of construction is a simple method of design. Part of the following work is devoted to calculations, including those required for shrinkage and creep. The theoretical development and exact solution of the relevant systems of differential equations are recorded in the author's works listed in the bibliography. With this information it is possible to produce rapid calculations for any project.

It would be misleading to lay down general rules for economy. The various conditions in the shop and on the site always lead to different methods of approach. The examples will demonstrate the wide range of application of composite construction.

Paper to be read before the Institution of Structural Engineers, at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 27th April, 1961, at 6 p.m.

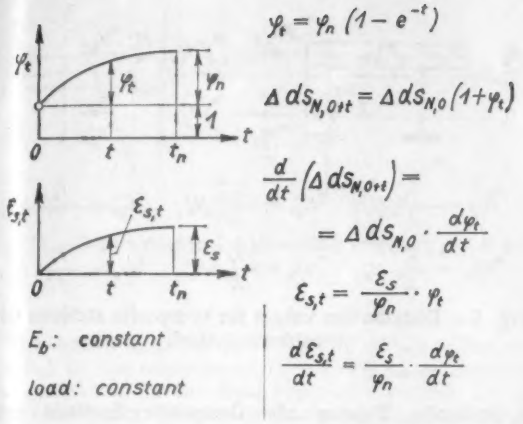

$\varphi_t = \varphi_n (1 - e^{-t})$
$\Delta ds_{N,0+t} = \Delta ds_{N,0}(1+\varphi_t)$
$\frac{d}{dt}(\Delta ds_{N,0+t}) = \Delta ds_{N,0} \cdot \frac{d\varphi_t}{dt}$
$\varepsilon_{s,t} = \frac{\varepsilon_s}{\varphi_n} \cdot \varphi_t$
$\frac{d\varepsilon_{s,t}}{dt} = \frac{\varepsilon_s}{\varphi_n} \cdot \frac{d\varphi_t}{dt}$
E_b: constant
load: constant

Fig. 3.—Variation of creep, elastic strain and shrinkage with time.

A. THEORETICAL CONSIDERATIONS

2. Creep and Shrinkage

The well-known relationship between elastic and plastic strain is shown in the upper diagram in Fig. 3 (see also Reference 1, p.3). At time t_n, the final creep factor

$$\varphi_n = \frac{\textit{plastic strain}}{\textit{elastic strain}}$$

From Whitney's idealised curves (see Reference 1, p.7), it can be concluded that the plastic strain during an interval of time dt is always proportional to the elastic strain due to all the loads applied up to the relevant time t times $d\varphi/dt$. The modulus of elasticity of concrete E_b used in the calculation is that existing at the initial application of load. If it is assumed that shrinkage $\varepsilon_{s,t}$ follows the same pattern as the creep curve φ_t, as also shown in Fig. 3, then a similar expression may also be derived for the strain due to shrinkage. The form of the φ_t and $\varepsilon_{s,t}$ curves has been confirmed in the author's tests on unreinforced concrete columns[3]. Similar results were obtained in deflection tests on reinforced concrete beams and slabs[3].

If the foregoing hypotheses are used, it is possible to represent the final strain after the completion of creep arising from any compressive load $N_{b,\varphi}$ on any concrete section as shown in Fig. 4. (See Reference 1, *inter alia* p. 78). If N_b is constant, as shown in Fig. 4a, the final elastic and plastic strain equals the elastic strain multiplied by $(1 + \varphi_n)$. In general, if $N_{b,\varphi}$ increases in accordance with any law of any kind, Fig. 4b applies. Three special cases are shown in Figs. 4c, d and e. If the compressive force N_b increases linearly, as in Fig. 4c, then the factor $\left(1 + \frac{\varphi_n}{2}\right)$ is used. If it is assumed that the growth of N_b varies with either $\frac{1 - e^{-\varphi_t}}{1 - e^{-\varphi_n}}$ or $\frac{1 - e^{-\beta\varphi_t}}{1 - e^{-\beta\varphi_n}}$ the factors are either $1 + \psi_b\varphi_n$ or $1 + \psi_{b,\beta}\varphi_n$ respectively. If the above principles are used, almost any load met in practice can be dealt with. The significance of these results will be shown later. The behaviour of members under flexure is similar.

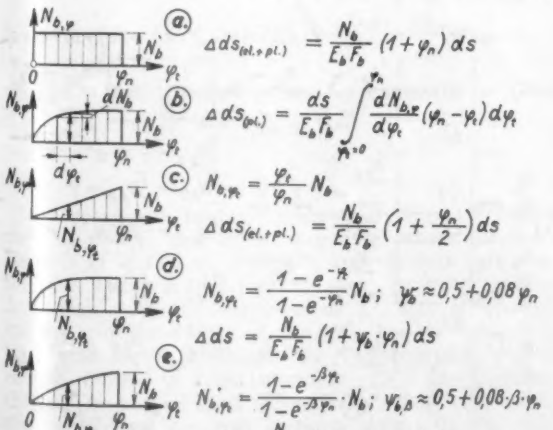

a.
$\Delta ds_{(el.+pl.)} = \frac{N_b}{E_b F_b}(1+\varphi_n)\,ds$
b.
$\Delta ds_{(pl.)} = \frac{ds}{E_b F_b}\int_{\varphi_t=0}^{\varphi_n} \frac{dN_{b,\varphi}}{d\varphi_t}(\varphi_n - \varphi_t)\,d\varphi_t$
c.
$N_{b,\varphi_t} = \frac{\varphi_t}{\varphi_n} N_b$
$\Delta ds_{(el.+pl.)} = \frac{N_b}{E_b F_b}\left(1 + \frac{\varphi_n}{2}\right) ds$
d.
$N_{b,\varphi_t} = \frac{1 - e^{-\varphi_t}}{1 - e^{-\varphi_n}} N_b$; $\psi_b \approx 0{,}5 + 0{,}08\,\varphi_n$
$\Delta ds = \frac{N_b}{E_b F_b}(1 + \psi_b \cdot \varphi_n)\,ds$
e.
$N_{b,\varphi_t} = \frac{1 - e^{-\beta\varphi_t}}{1 - e^{-\beta\varphi_n}} N_b$; $\psi_{b,\beta} \approx 0{,}5 + 0{,}08\,\beta \cdot \varphi_n$
$\Delta ds = \frac{N_b}{E_b F_b}(1 + \psi_{b,\beta}\,\varphi_n)\,ds$

Fig. 4.—Strains induced by various longitudinal forces.

3. Idealised Cross-Sections

On the assumption that plane sections remain plane after bending, any given composite beam behaves like a homogeneous transformed or idealised section in terms of structural steel possessing reduced cross-sectional areas $F_{b,r}$ and $F_{z,r}$ and reduced moments of inertia $J_{b,r}$ and $J_{z,r}$ when the two corresponding moduli of elasticity for concrete E_b and prestressing steel E_z are related to that of structural steel E_e through the respective modular ratios n_b and n_z.

For a section without prestressing steel, the idealised values F_i, S_i and J_i, together with the redistribution factor α_{st}, are as shown in Fig. 5.

In a similar fashion, for a section embodying prestressing steel, the total steel section (beam and prestressing steel) can be expressed in terms of area F_t and moment of inertia J_t, as shown in Fig. 6, from which idealised values F'_i, J'_i, S'_i and α'_{st}, corresponding to those in Fig. 5, can be derived.

Any composite section can be transformed into an idealised section. Fig. 7 gives some redistribution factors α for various kinds of cross-section which, as will be shown later, simplify the calculations for the influence of creep and shrinkage. Formulas for any term in any kind of cross-section are tabulated in Reference 1.

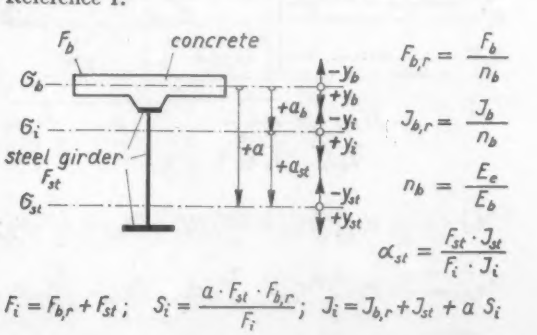
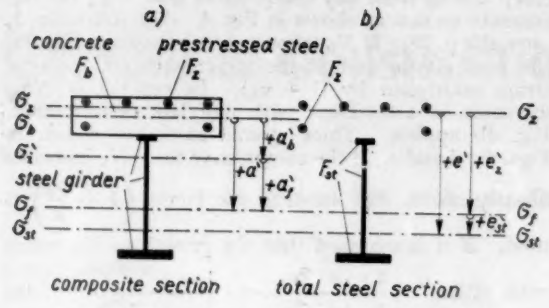


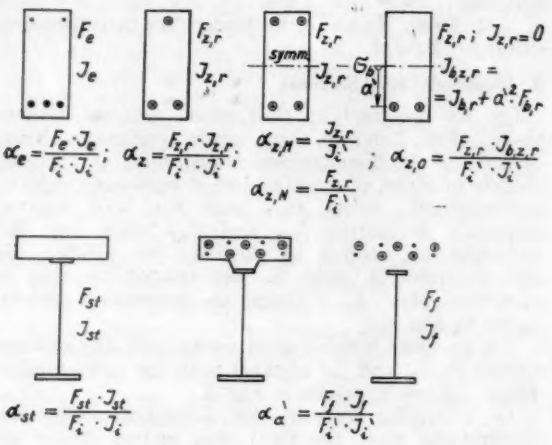
Fig. 5.—Idealised values for transformed section without prestressing steel.



$$F_{z,r} = \frac{F_z}{n_z}; \quad J_{z,r} = \frac{J_z}{n_z}; \quad n_z = \frac{E_e}{E_z}$$

$$F_f = F_{st} + F_{z,r}; \quad J_f = J_{st} + J_{z,r} + e' S_f$$

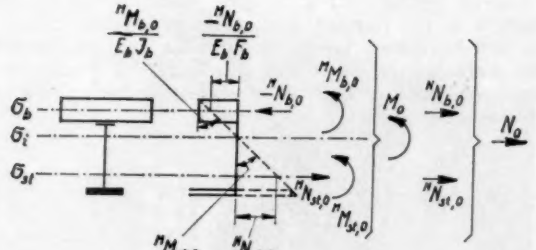
Fig. 6.—Idealised steel section (beam + prestressing steel).



$$\alpha_e = \frac{F_e \cdot J_e}{F_i \cdot J_i}; \quad \alpha_z = \frac{F_{z,r} \cdot J_{z,r}}{F_i \cdot J_i}; \quad \alpha_{z,M} = \frac{J_{z,r}}{J_i}; \quad \alpha_{z,N} = \frac{F_{z,r}}{F_i}; \quad \alpha_{z,0} = \frac{F_{z,r} \cdot J_{b,z,r}}{F_i \cdot J_i}$$

$$\alpha_{st} = \frac{F_{st} \cdot J_{st}}{F_i \cdot J_i} \qquad \alpha_\alpha = \frac{F_f \cdot J_f}{F_i \cdot J_i}$$

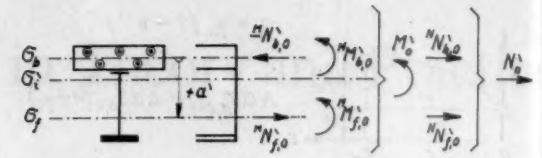
Fig. 7.—Redistribution factors for various sections.



$$^M N_{b,0} = -\frac{S_i}{J_i} M_0; \quad ^M M_{b,0} = \frac{J_{b,r}}{J_i} M_0; \quad ^N N_{b,0} = \frac{F_{b,r}}{F_i} N_0;$$

$$^M N_{st,0} = \frac{S_i}{J_i} M_0; \quad ^M M_{st,0} = \frac{J_{st}}{J_i} M_0; \quad ^N N_{st,0} = \frac{F_{st}}{F_i} N_0.$$

Fig. 8.—Distribution values for composite sections without prestressing steel.



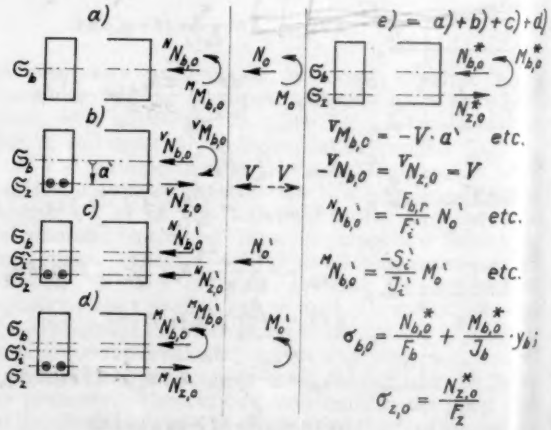
$$^M N'_{b,0} = -\frac{S'_i}{J'_i} M'_0; \quad ^M M'_{b,0} = \frac{J_{b,r}}{J'_i} M'_0; \quad ^N N'_{b,0} = \frac{F_{b,r}}{F'_i} N'_0;$$

$$^M N'_{f,0} = \frac{S'_i}{J'_i} M'_0; \quad ^M M'_{f,0} = \frac{J_f}{J'_i} M'_0; \quad ^N N'_{f,0} = \frac{F_f}{F'_i} N'_0.$$

Fig. 9.—Distribution values for composite sections with prestressing steel.

4. Statically Determinate Composite Sections with Solid Webs

If a longitudinal force N_0 and a moment M_0 are applied at the centroid G_1 of a composite section, it is possible by using the normal conditions of equilibrium and compatibility to distribute the resultant forces between the concrete and steel in accordance with the respective areas of the transformed section, as shown in Figs. 8 and 9 respectively for sections without and with prestressing steel. In Reference 1 there are formulas for distribution factors for every possible condition of loading and cross-section, arranged in sequence according to the occurrence of the individual loads. Such loads can be applied to various parts of the section, for example, concrete can be placed on an unsupported or only a partially-propped steel beam, its weight being taken by the beam alone (F_{st}, J_{st}). When props are removed, the initial prop loads no longer apply to the composite section (F_1, J_1), neither does the prestressing induced by them. A load, however, such as an asphalt carpet, which is applied after prestressing tendons have been tensioned, acts on the whole cross-section (F'_1, J'_1). By summating the values for all the individual conditions, the final values $N^*_{b,0}$, $M^*_{b,0}$, $N^*_{st,0}$ etc., are obtained. In Fig. 10, simple examples are given for prestressed concrete sections in which the moment of inertia of the prestressing steel about its own axis (i.e. the self-inertia) has been ignored (i.e. $J_{z,r} = 0$). Here, the longitudinal force N_0, moment M_0 and prestressing force V act on the concrete section alone, while N'_0 and M'_0 act on the composite section.



$$^V M_{b,0} = -V \cdot \alpha' \quad \text{etc.}$$

$$-^V N_{b,0} = {}^V N_{z,0} = V$$

$$^N N'_{b,0} = \frac{F_{b,r}}{F'_i} N'_0 \quad \text{etc.}$$

$$^M N'_{b,0} = \frac{-S'_i}{J'_i} M'_0 \quad \text{etc.}$$

$$\sigma_{b,0} = \frac{N^*_{b,0}}{F_b} + \frac{M^*_{b,0}}{J_b} \cdot y_b;$$

$$\sigma_{z,0} = \frac{N^*_{z,0}}{F_z}$$

Fig. 10.—Distribution values when self-inertia of steel is ignored.

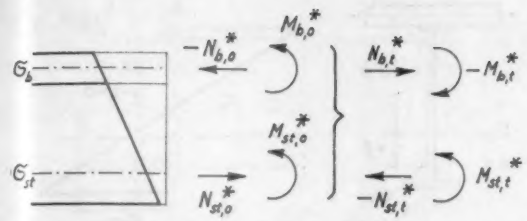


Fig. 11.—Strain during $d\varphi_t$ in section where $J_{z,r} \neq 0$ and $j_{st} < 0{\cdot}2$.

The distribution values $N^*_{b,0}$ and $M^*_{b,0}$ which are applied to the concrete at time $t = 0$ cause creep in the concrete. In the final composite section, i.e. at time t, the concrete deforms due to creep and shrinkage and tends to shed its load on to the steel section. The corresponding forces on the concrete and steel sections are shown as redistribution values $N^*_{b,t}$, $M^*_{b,t}$, $N^*_{st,t}$, $M^*_{st,t}$, etc., in Fig. 11.

For these four new values, there are also two conditions of equilibrium and two of compatibility. Using the principles deduced in section 2, it may be shown that for any increment of time dt and increment of creep $d\varphi_t$ two differential equations may be derived for the values of $N^*_{b,t}$ and $M^*_{b,t}$ in the concrete. (See Reference 1, p. 83). From research[1], it may be assumed that the modulus of elasticity E_b of the concrete is constant.

In fundamental investigations it is necessary to produce a rigorous solution (Reference 1, p. 85) as otherwise it is not possible to derive approximate solutions or prescribe limits of validity. However, in practice, such rigorous solutions are tedious.

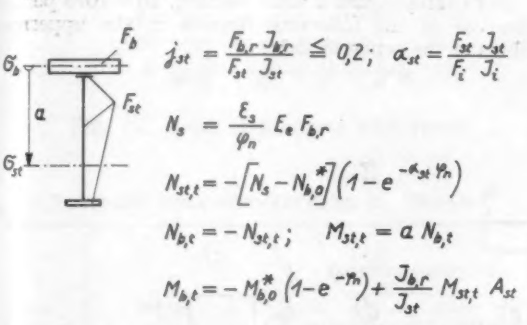


Fig. 12.—Redistribution values for composite sections where $j_{st} \leqq 0{\cdot}2$.

For composite beams, as shown in Fig. 12, where the criterion $j_{st} \leqq 0{\cdot}2$, i.e. in general where the steel beam is at least 1 m. deep, the approximate formulas for the redistribution values are accurate within 2 per cent. As the values of $N^*_{b,0}$ and $M^*_{b,0}$ are known and the values $[1 - e^{-\varphi_n}, 1 - e^{-\alpha_{st}\varphi_n}$ and $A_{st}]$ have been tabulated, the redistribution values can be derived in a few minutes. They are no more difficult to obtain than the corresponding values for the prestressed concrete sections shown in Fig. 13, where the self inertia of the prestressing steel is ignored ($J_{z,r} = 0$). The influence of shrinkage is calculated by using the value N_s.

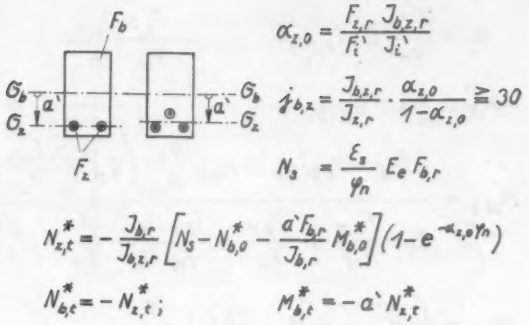


Fig. 13.—Redistribution values for prestressed concrete sections where $j_{b,z} = 30$ and $J_{z,r} \geqq 0$.

For composite beams where the criterion $j_{st} \geqq 0{\cdot}2$, i.e. in shallow construction, the formulas given in Fig. 12 are no longer valid. Considering, not the creep interval $d\varphi_t$, but the final deformation, as in Fig. 4, then the plastic deformation is φ times greater than the elastic but the redistribution values are $\psi \cdot \varphi$ times as great. The linear equations in Fig. 14 have been derived for these conditions. From the results of exhaustive tests, for $j_{st} \geqq 0{\cdot}2$, ψ can be taken as $0{\cdot}65$, from which the redistribution values for this group of beams have been derived as shown in Fig. 15 (Reference, 1, p.101).

For prestressed concrete sections in which the self inertia of the steel is included, the formulas in Fig. 13 can no longer be used when the criterion $j_{b,z} \leqq 30$. In this case the formulas in Fig. 15 apply but with $\Phi_{st} = 1 + \psi_{z,0}\varphi_n$ and $\psi_{z,0} = 0{\cdot}5 + 0{\cdot}8\alpha_{z,0}\varphi_n$ (Reference 1, p. 105). From Fig. 16 it will be seen that for cross sections e and d the formulas for $j_b \leqq 30$ must be used. This is important in prestressed bridge decks.

The stresses in the steel and concrete can be obtained from the distribution and redistribution values, not only initially when $t = 0$ but also at any later time t_n, from the well-known relationship

$$\sigma = \frac{N}{F} + \frac{M}{Z}$$

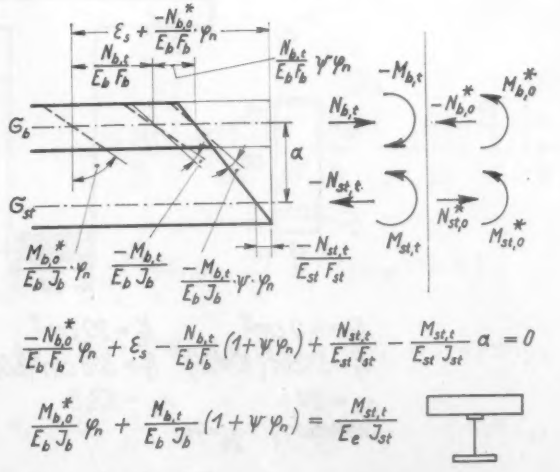


Fig. 14.—Calculation of the redistribution values for final strain.

$$N_S = \frac{\varepsilon_s}{\varphi_n} E_e F_{b,r}; \qquad j_{st} = \frac{F_{b,r}}{F_{st}} \frac{J_{b,r}}{J_{st}}$$

$$\Phi_{st} = 1 + 0{,}65\,\varphi_n$$

$$M_{st,t} = \frac{a(N_S - N_{b,0})\varphi_n + M_{b,0}\frac{\varphi_n}{\Phi_{st}}\left(\Phi_{st} + \frac{F_{b,r}}{F_{st}}\right)}{\Phi_{st} + \frac{F_{b,r}}{F_{st}} + \frac{J_{b,r}}{J_{st}} + \frac{F_{b,r}}{F_{st}}\frac{J_{b,r}}{J_{st}}\frac{1}{\Phi_{st}} + a^2\frac{F_{b,r}}{J_{st}}}$$

$$M_{b,t} = \left(-M_{b,0}\varphi_n + M_{st,t}\frac{J_{b,r}}{J_{st}}\right)\frac{1}{\Phi_{st}}$$

$$N_{st,t} = -\frac{1}{a}(M_{st,t} + M_{b,t}); \qquad N_{b,t} = -N_{st,t}$$

Fig. 15.—Redistribution values for composite sections where $j_{st} \geqq 0{\cdot}2$.

Fig. 17 shows the redistribution of stresses due to creep and shrinkage. Although there is little change in the stresses on the underside of the beam it should be noted that the pattern changes in the web. This is important when the buckling strength of the web is being considered. The existence of a so-called neutral axis of creep (KN) will also be noted, which is unaffected by shrinkage or creep and the position of which can be derived from Fig. 17 (Reference 1, p. 143).

From the constitution of the above systems of equations it can be established that for composite structures where the self inertia of the steel is appreciable all formulas calculated with the idealised value n_i cannot be accurate and, in fact, can lead to appreciable errors on either side. One idealised value cannot be used in the solution of two differing conditions of compatibility.

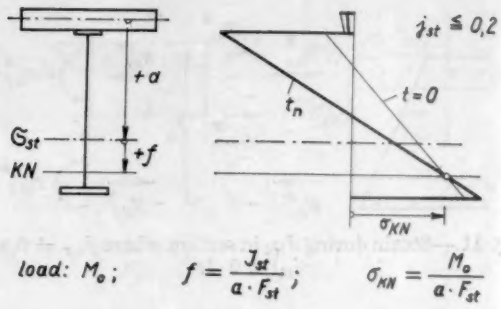


Fig. 17.—Stress redistribution and the neutral axis for creep.

It is appropriate to give an idea of the magnitude of the individual influences already mentioned. Fig. 18 shows, for example, the magnitude of the tensile forces to be expected from shrinkage in the concrete slab relative to given redistribution factors α_{st} and coefficients of shrinkage ε_s. Note that for a normal structure $\alpha_{st} = 0{\cdot}05$ and $\varphi_n = 4{\cdot}0$ while for an average bridge $\alpha'_{st} = 0{\cdot}2$ and $\varphi_n = 2{\cdot}0$.

Fig. 19 shows the creep loss in prestressing relative to α'_{st} and φ_n. If one considers a creep loss of more than 30 per cent as uneconomical, it can be seen for example that prestressing in a structure with $\alpha'_{st} = 0{\cdot}3$ and $\varphi_n = 4{\cdot}0$ is undesirable as the creep loss is 70 per cent.

It can be seen from Figs. 18 and 19 that if one wants to be economical one should never prestress more than necessary and one should make the greatest possible use of the permissible tensile stresses allowed in the concrete under the final condition t_n.

For example, under main loading, DIN 1078 permits the use of the following stresses in the uppermost fibres of the concrete slab :

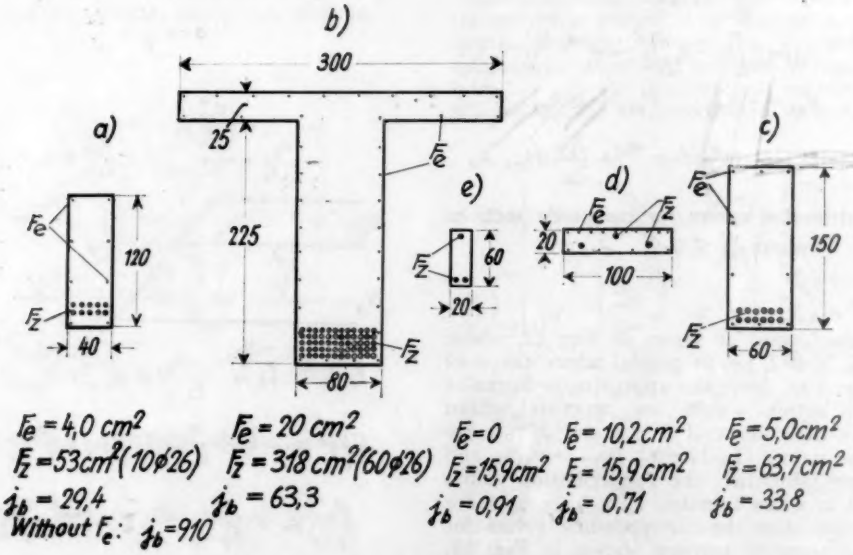


Fig. 16.—Comparison of j_b values in prestressed concrete sections.

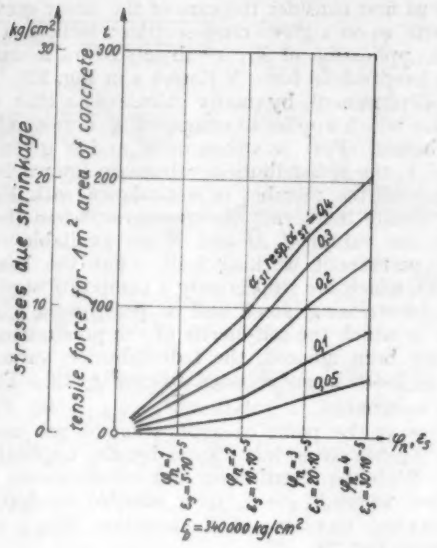


Fig. 18.—Shrinkage stresses.

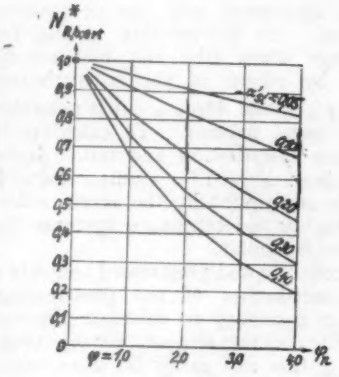


Fig. 19.—Loss of prestress with creep.

Permissible tensile stresses in kg/sq.cm. (lb/sq.in.)

Case	Bridge deck	Concrete quality at 28 days	
		B 300 (4267)	B 600 (8534)
1	Unprotected	0	18 (250)
2	Asphalt carpet	20 (278)	30 (417)
3	Asphalt carpet and isolation	25 (347)	35 (487)

Case **3** is the most economical as it involves the least possible amount of prestressing. In consequence, most bridges comply with Case 3.

Schrader's book[6] can be recommended for the preliminary design of composite girders, both with and without prestressing.

The effect of differences of temperature between the concrete slab and steel beam can be established from the conditions of equilibrium and compatibility shown in Fig. 20. Simple formulas exist for the

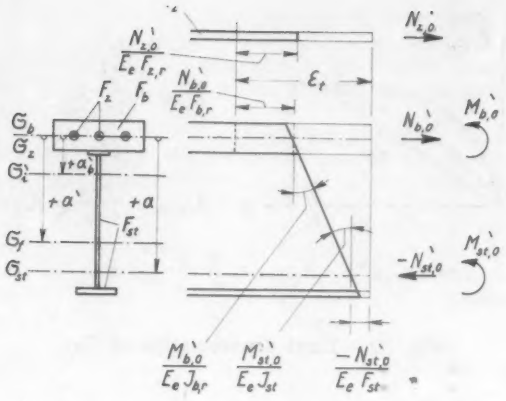
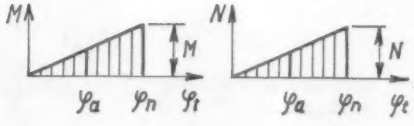


Fig. 20.—Distribution values arising from temperature variation in concrete.

appropriate distribution values (Reference 1, p. 240). It should be noted that the coefficient of expansion of limestone concrete can differ appreciably from that of steel.

The foregoing calculations for the redistribution values were based on constant loads unvaried by the passage of time. Loads can also occur which vary according to some determinable law from zero at $t = 0$ to final values at time t_n. It is evident that these final values must be smaller than those applicable to constantly applied loads. Here again, it has been possible to establish simple approximate formulas after rigorous investigation. In Fig. 21 the values are derived for a load due to creep which increases linearly with φ_t. Accordingly, the values for $N_{st,t}$, $N_{b,t}$ and $M_{st,t}$ for a constant load need only be multiplied by a reduction factor λ_{st}. B_{st} and C_{st} are already tabulated. In a similar manner, reduction factors have been developed for any given cross-section and for various laws governing the growth of the given load (Reference 1, pp. 107-142). Among other things, it is possible with these formulas to determine the effect of the movement of piers during the passage of time.



$$N_{st,t} = \lambda_{st}\left[\frac{F_{b,r}}{F_i} N - \frac{S_i}{J_i} M\right]\left(1 - e^{-\alpha_{st}\varphi_n}\right)$$

$$\lambda_{st} \approx 0{,}5 + 0{,}08\,\alpha_{st}\,\varphi_n;\quad N_{b,t} = -N_{st,t};\quad M_{st,t} = -a\,N_{st,t}$$

$$M_{b,t} = a\left[\frac{F_{b,r}}{F_i} N - \frac{S_i}{J_i} M\right]\frac{J_{b,r}}{J_{st}} B_{st} - \frac{J_{b,r}}{J_i} M\, C_{st}$$

Fig. 21.—Redistribution values for sections in which $j_{st} \leq 0{\cdot}2$.

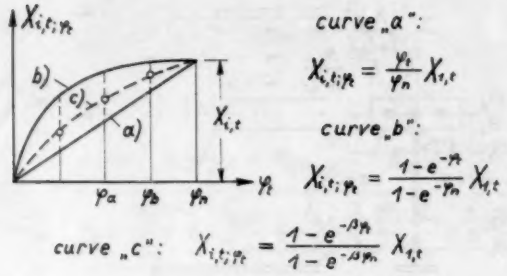


Fig. 22.—Final determination of $X_{1,t}$.

5. Statically Indeterminate Composite Sections with Solid Webs

As deformations due to creep and shrinkage appear in a statically indeterminate structure during the passage of time, new unknown forces $X_{1,t}$ must be induced. On the one hand, these depend upon the shrinkage of the concrete slab and upon the creep induced in the same by the given loads (permanent loads, prestressing, etc.), and on the other upon the redundant forces themselves and the creep which they, in turn, induce. Although these complicated conditions can be expressed in the form of differential equations, the latter can only be solved for certain simple cases and, in consequence, they are not used in practice. But as the growth of $X_{1,t}$ with time and with φ_t respectively follows a definite pattern from the very outset, as shown in Fig. 22, it may be assumed that the forces induced in all the various cross-sections follow the same law. Therefore, the appropriate redistribution values can be determined as in section 4, together with the corresponding deformations. As will be shown, with this basis of analysis the appropriate values of $X_{1,t}$ can be obtained in a similar manner to that employed for the evaluation of the redundants X_1 in any other statically indeterminate structure. After completion of the calculations one may check whether the selected course of $X_{1,t}$ coincides with the value obtained by calculation. This need only be done for a few values of φ_t. If there is no agreement between them, in most cases it is possible to adjust the values to the actual conditions by a few slide-rule calculations.

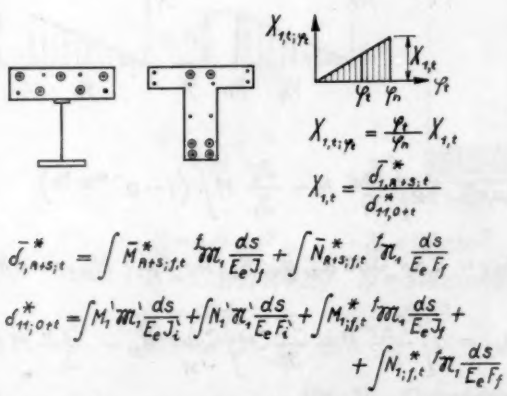


Fig. 23.—Determination of $X_{1,t}$.

Let us first consider the case of the linear growth of $X_{1,t}$ with φ_t on a given cross-section which, as a result of the application of X_1, is subjected to a moment M and a longitudinal force N (Curve *a* in Fig. 22). It has been determined[1] by many calculations that this is the case which applies to composite girders comprising steel beams. For the values of M and N given when $X_{1,t} = 1$, the redistribution values are known for any position in the member in accordance with Fig. 21. In statically indeterminate structures, when the time $t = 0$, the values of $\bar{M}$ and $\bar{N}$ are available for the given permanent working load. For the beams in Fig. 23, which are respectively a composite steel beam with prestressing steel and a prestressed concrete beam in which the self-inertia of the prestressing steel has not been ignored, the redistribution values $\bar{M}_{t,t}$ and $\bar{N}_{t,t}$ can be established from Fig. 12. Through their occurrence, a rotation $\bar{\delta}^*_{1,R+s;t}$ (see Fig. 25) appears at the point of application of the unknown force X_1, which is only closed by the application of $X_{1,t}$. With the distribution and redistribution values induced when $X_{1,t} = 1$, it is possible to derive the appropriate increases in deformation $\delta^*_{11,0+t}$ as shown in Fig. 23.

The newly appearing unknown $X_{1,t}$ is composed of distribution values (first two terms) and of redistribution values. As the deformation of the steel section must be in agreement with the deformation of the whole section, the deformation arising from creep and shrinkage alone (the last two terms) can be determined by means of the redistribution values. With $\bar{\delta}^*_{1,R+s;t}$ and $\delta^*_{11,0+t}$ it is possible to obtain $X_{1,t}$ in the usual manner. To calculate the actual stresses, those distribution and redistribution values which have been derived for a unit value of $X_{1,t}$ must, of course, be multiplied by the actual values of $X_{1,t}$. For more highly indeterminate systems the method of calculation is similar.

In plain concrete and prestressed concrete structures where the self-inertia of the prestressing steel is ignored, it is necessary to calculate the variation of stress over the concrete section. By selecting a suitable law for $X_{1,t}$, this can easily be done with the help of Fig. 4. For a tied concrete arch, as in Fig. 24, the plastic deformations (due to creep) can be determined merely by multiplying the elastic deformations by φ_n and $\psi_b \cdot \varphi_n$ respectively.

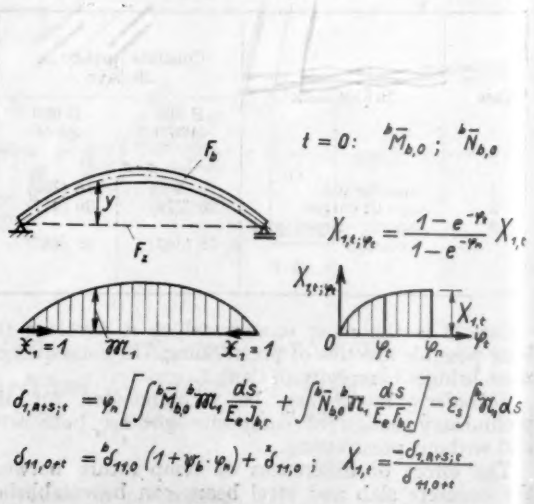
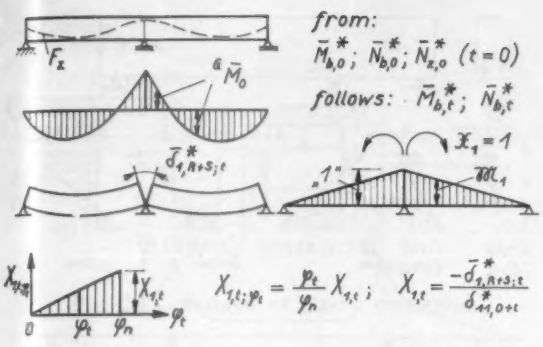


Fig. 24.—Determination of $X_{1,t}$ for arch.



$$X_{1,t;\varphi_t} = \frac{\varphi_t}{\varphi_n} X_{1,t}\,; \qquad X_{1,t} = \frac{-\bar{\delta}^*_{1,R+S;t}}{\delta^*_{11,0+t}}$$

$$\bar{\delta}^*_{1,R+S;t} = \int \bar{M}^*_{b,0}\, {}^{b}\overline{m}_1 \frac{\varphi_n}{E_e J_{b,r}}\, ds + \int \bar{M}^*_{b,t}\, {}^{b}\overline{m}_1 \frac{(1+\psi_{z,0}\varphi_n)}{E_e J_{b,r}}\, ds$$

$$\delta^*_{m,0+t} = \left(1+\frac{\varphi_n}{2}\right)\cdot \int M^*_{1;b,0}\, {}^{b}\overline{m}_1 \frac{ds}{E_e J_{b,r}} + \left(1+\psi^+_{z,0}\varphi_n\right)\cdot \int M^*_{1;b,t}\, {}^{b}\overline{m}_1 \frac{ds}{E_e J_{b,r}}$$

$$\psi_{z,0} \approx 0{,}5 + 0{,}08\,\alpha_{z,0}\,\varphi_n\,; \qquad \psi^+_{z,0} \approx 0{,}34$$

Fig. 25.—Determination of $X_{1,t}$ for prestressed concrete section.

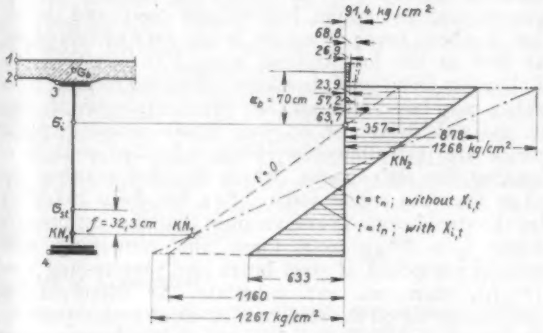


Fig. 26.—Stress distribution.

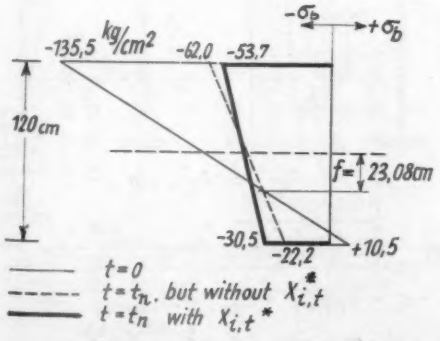


Fig. 27.—Stress redistribution through creep and shrinkage.

It will also be observed in Fig. 25 that, with the help of $\psi_{z,0}$ and $\psi^+_{z,0}$, the unknown $X_{1,t}$ can be quickly and easily evaluated for prestressed concrete beams in which the self-inertia of the prestressing steel is ignored.

To demonstrate the important influence of the unknown $X_{1,t}$ in composite steel beams, Fig. 26 shows the stress distribution in a cross-section of a continuous girder having spans of 60 + 90 + 60 m.

For many years, the stress measurements which have been recorded in the continuous trussed span motor-road bridge at Montabaur[7] have revealed close agreement with the calculations made in accordance with the foregoing methods of design.

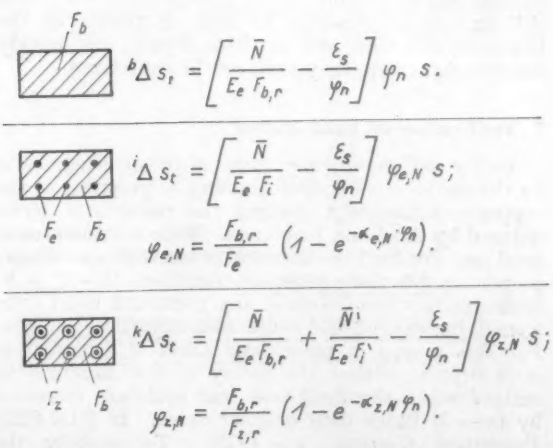
The prestressed concrete example in Fig. 27 is included to demonstrate that in such structures the influence of $X_{1,t}$ is often very small.

6. Composite Construction with Lattice Girders

Lattice girders can be used as internal main girders, as cross-beams when the main girders are very widely-spaced, as diaphragms and as stringers. When the concrete is in direct contact with any special steel top chord, the calculations are increased because creep and shrinkage are hindered and, in consequence, heavy stresses are induced. *Bandel* [8] has treated this case exhaustively. If, however, such a chord is used only temporarily to facilitate erection or to support the shuttering, then the pattern of stress is much more favourable and the calculation more simple.

In statically determinate construction the creep and shrinkage of the concrete slab create deformation but no other form of stress. If untensioned reinforcement or prestressing tendons are used, and these are arranged symmetrically, then the longitudinal variation of the length S of the concrete slab can be determined from Fig. 28, wherein the creep factors, φ_n, for unreinforced concrete, $\varphi_{e,N}$, for reinforced concrete and $\varphi_{z,N}$, for prestressed concrete, are used. If the variation in length of the individual members due to any load is known, in concrete including creep and shrinkage, then the deformation of the whole lattice girder can be calculated in the usual manner. $\bar{N}$ and $\bar{N}^1$ are the compressive forces in any concrete section, respectively without and with prestressing tendons.

In statically indeterminate girders, as in Fig. 29, the increases in strain $\delta_{u,R+S;t}$ from shrinkage and creep due to the permanent loads at the points



$${}^{b}\Delta s_t = \left[\frac{\bar{N}}{E_e F_{b,r}} - \frac{\varepsilon_s}{\varphi_n}\right]\varphi_n\, s.$$

$${}^{i}\Delta s_t = \left[\frac{\bar{N}}{E_e F_i} - \frac{\varepsilon_s}{\varphi_n}\right]\varphi_{e,N}\, s;$$

$$\varphi_{e,N} = \frac{F_{b,r}}{F_e}\left(1 - e^{-\alpha_{e,N}\cdot\varphi_n}\right).$$

$${}^{k}\Delta s_t = \left[\frac{\bar{N}}{E_e F_{b,r}} + \frac{\bar{N}^1}{E_e F_i} - \frac{\varepsilon_s}{\varphi_n}\right]\varphi_{z,N}\, s;$$

$$\varphi_{z,N} = \frac{F_{b,r}}{F_{z,r}}\left(1 - e^{-\alpha_{z,N}\varphi_n}\right).$$

Fig. 28.—Strain due to creep and shrinkage.

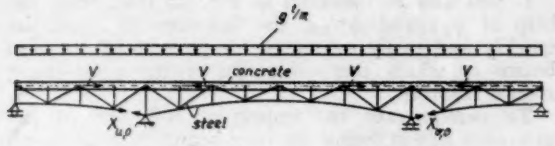


Fig. 29.—Latticed composite girder without a steel top chord.

of application of the statically indeterminate forces can be derived from Fig. 28. If one again selects the law of increase with time and with φ_t for the unknown $X_{u,t}$, then the increase of strain at the point of application of these statically indeterminate values—elastic and plastic—can be calculated as before. On the basis of a linear law for the values $X_{u,t}$, which usually applies to latticed composite girders, these increases of strain can be calculated in accordance with Fig. 30. For plain concrete sections, the creep factor $\varphi_n/2$ should be used, while the factors $\varphi_{e,N}$ and $\varphi_{z,N}$ should be used for untensioned and prestressed steel respectively. The unknowns are calculated in the usual manner.

$$\delta_{u,\beta+s;t} = \sum {}^{b}\Delta S_t \; {}^{t}\mathfrak{N}_{\beta} + \sum {}^{i}\Delta S_t \; \mathfrak{N}_u + \sum {}^{k}\Delta S_t \; \mathfrak{N}'_u$$

$$\delta_{uv,\,0+t} = \sum {}^{b}N_u \; {}^{b}\mathfrak{N}_v \; \frac{1+\frac{\varphi_n}{2}}{E_e \, F_{b,r}} + \sum N_u \; \mathfrak{N}_v \; \frac{1+\varphi^{+}_{e,N}}{E_e \, F_i} \, s +$$

$$+ \sum N'_u \; \mathfrak{N}'_v \; \frac{1+\varphi^{+}_{z,N}}{E_e \, F'_i} \, s$$

$$\varphi^{+}_{e,N} = \frac{F_{b,r}}{F_e} \left(1 - e^{-\alpha_{e,N}\varphi_n}\right)\left(0{,}5 + 0{,}08\,\alpha_{e,N}\,\varphi_n\right)$$

$$\varphi^{+}_{z,N} = \frac{F_{b,r}}{F_{z,r}} \left(1 - e^{-\alpha_{z,N}\varphi_n}\right)\left(0{,}5 + 0{,}08\,\alpha_{z,N}\,\varphi_n\right)$$

Fig. 30.—Terms for statically indeterminate lattice girder.

In view of its great rigidity, heavy secondary stresses can be created in the concrete slab while it is still shuttered. However, no traffic is allowed on the bridge at this time and as these stresses are quickly lost through creep, they need not be considered.

7. Verification of Load Factor

As Fig. 31 shows, any form of prestressing, be it by the use of steel beams, trussing or propping of the supports, completely changes the pattern of stress induced by dead and live loads. With any increase of dead and live load the increase in stress Z is no longer proportional to the total stress conditions $G + P + V$. Under certain conditions at any particular point even a small increase of load could lead to yield or collapse. For this reason in prestressed concrete construction it is stipulated that the safety of the structure be verified when the dead and live loads are increased by $(\nu - 1)$ times their original value. In DIN 4227, *Prestressed Concrete*, $\nu = 1{\cdot}75$. To simplify the calculations such a check is not made with prestressed composite steel beams, but instead the load factor

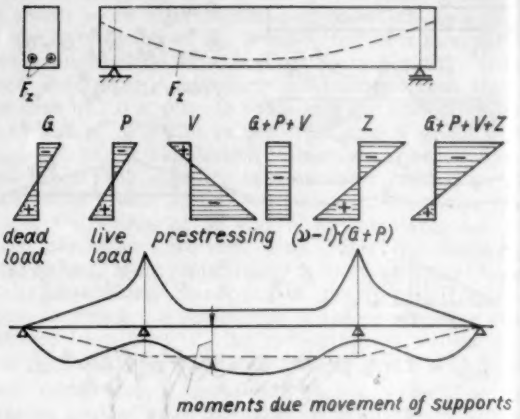
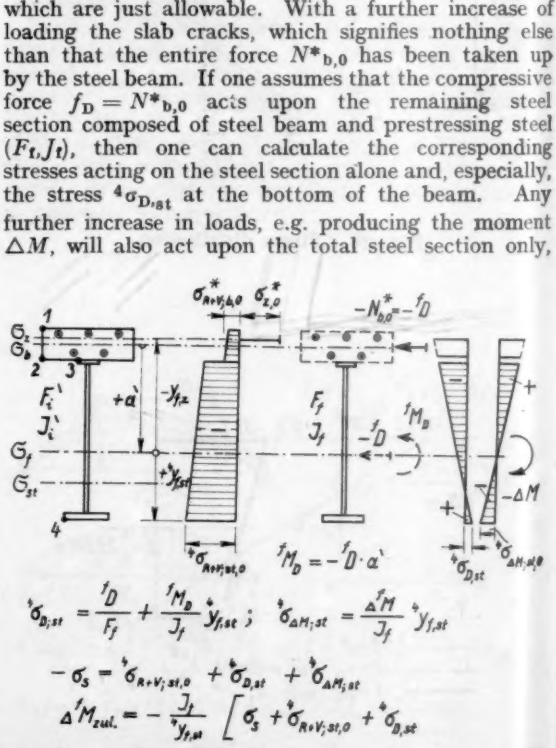


Fig. 31.—The effect of prestressing on stress conditions.

against yield in the steel beam is verified, the minimum value of ν required by DIN 1078 being 1·6. In these calculations the stress analyses already made for the normal working loads can be used again. The stresses due to creep and shrinkage alone are of no interest as they have no practical influence on collapse. The check for statically determinate prestressed sections is shown in Fig. 32. The stress ${}^{4}\sigma_{R+V;\,st,0}$ at point 4 on the bottom flange due to dead loads, prestressing and traffic, but without creep and shrinkage, has been found as set out in section 4 of this paper, as well as the longitudinal force $N^*_{b,0}$ in the slab. Following creep and shrinkage, the longitudinal force $N^*_{b,0}$ may be so reduced that the compressive stresses in the slab disappear or even tensile stresses appear which are just allowable. With a further increase of loading the slab cracks, which signifies nothing else than that the entire force $N^*_{b,0}$ has been taken up by the steel beam. If one assumes that the compressive force $f_D = N^*_{b,0}$ acts upon the remaining steel section composed of steel beam and prestressing steel (F_t, J_t), then one can calculate the corresponding stresses acting on the steel section alone and, especially, the stress ${}^{4}\sigma_{D;st}$ at the bottom of the beam. Any further increase in loads, e.g. producing the moment ΔM, will also act upon the total steel section only,



$$f_{M_D} = -f_D \cdot \alpha'$$

$${}^{4}\sigma_{D;st} = \frac{f_D}{F_f} + \frac{f_{M_D}}{J_f}\,{}^{4}y_{f,st}\,; \qquad {}^{4}\sigma_{\Delta M;st} = \frac{{}^{f}\Delta M}{J_f}\,{}^{4}y_{f,st}$$

$$-\sigma_s = {}^{4}\sigma_{R+V;\,st,0} + {}^{4}\sigma_{D,st} + {}^{4}\sigma_{\Delta M;st}$$

$${}^{f}\Delta M_{zul.} = -\frac{J_f}{{}^{4}y_{f,st}}\left[\sigma_s + {}^{4}\sigma_{R+V;\,st,0} + {}^{4}\sigma_{D,st}\right.$$

Fig. 32.—Verification of load factor.

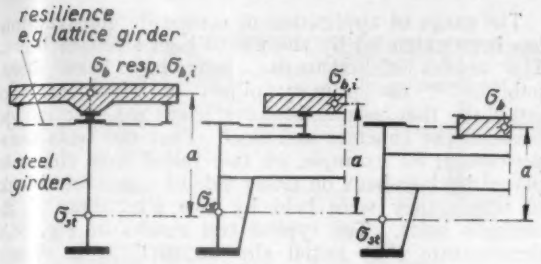


Fig. 33.—Methods of elastic connexion.

producing the stresses shown. The sum of all the stresses at point 4 may reach the yield stress σ_s. From this condition, the single unknown value ΔM can clearly be established and, in consequence, the increase of loading and the load factor. For statically indeterminate systems, the process is similar.

In concluding this section, it may be stated that the verification of load factor is very rarely critical in design.

8. Shear Forces and Connectors.

If, for any given working conditions, with and without creep and shrinkage, the longitudinal forces N_{st} acting on a steel beam are known, the difference in force between any two given cross-sections must be absorbed by shear forces, the evaluation of which may be readily carried out for both $t = 0$ and $t = t_n$, by considering the given values of N_{st}.

So much research has been undertaken and so many papers published on the carrying capacity, design and construction of shear connectors (see Reference 1, p. 234; DIN 1078 and DIN 4239) that little more could be added here. It may suffice to state that large factors of safety are inherent in the use of the rigid dowel type of connector.

9. Elastic Connexion

For economic reasons elastic connexion, shown diagrammatically in Fig. 33, is normally used only in special circumstances, e.g. in the Cologne-Rodenkirchen suspension bridge over the Rhine, which was built in 1954 and which has a central span of 378 m, where despite the large deflections induced in such a structure, a completely jointless deck was constructed over the

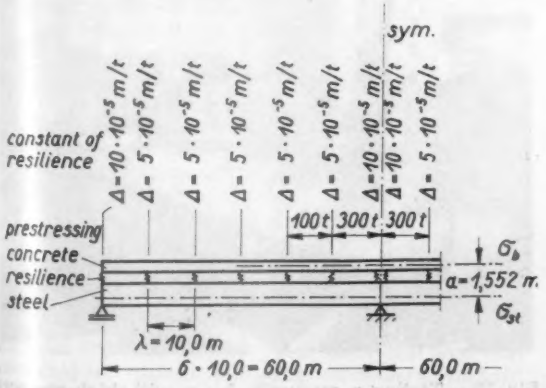


Fig. 34.—Elastic connexion in two-span girder.

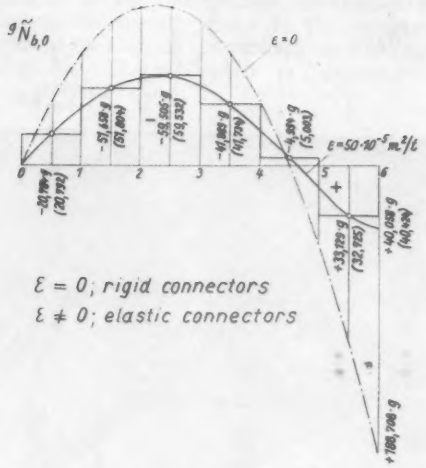


Fig. 35.—Forces in two-span girder with elastic connexion.

entire length of 580 m of the bridge. The idea of elastic connexion stems from a desire to disengage the concrete slab and steel beam in the zone of large negative support moments, thereby reducing the prestressing forces required in the concrete slab to a minimum. To make such a system of elastic connexion as simple as possible, the author has devised a method of calculation by which the technique can be restricted to any desired portion of the structure[9]. The time consumed in calculation, both with and without creep and shrinkage, is quite modest. For creep, the variation of redistribution values in the slab with time and φ_t respectively may be assumed to follow the previously mentioned pattern.

Consider, for example, the continuous composite girder shown in Fig. 34 in which for simplicity of calculation the suspension has been restricted to positions at 10 m centres along the girder. The longitudinal forces $N_{b,0}$ induced in the concrete slab at time $t = 0$ by a uniformly distributed load g are shown in Fig. 35, the figures in brackets, obtained by a more rigorous solution, being included for comparison. It will be observed that the longitudinal forces in the slab are appreciably less than those calculated for conditions with completely rigid connexion. Although this arrangement is favourable over the support, it is not economical at mid-span where the great reserve of strength of the slab is not being fully utilised.

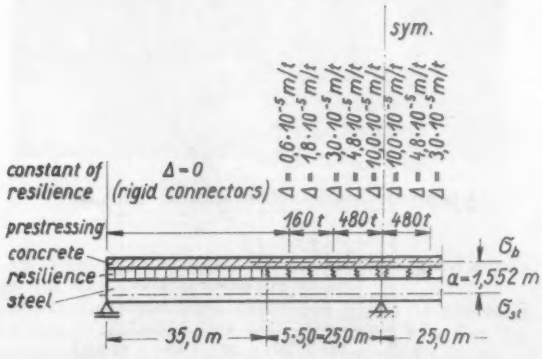


Fig. 36.—Rigid and elastic connexion in two-span girder.

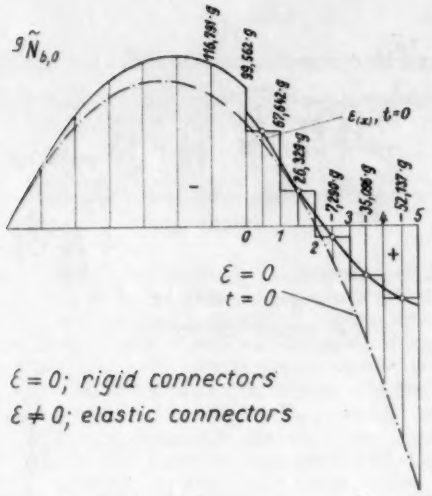


Fig. 37.—Forces in two-span girder with rigid and elastic connexion.

However, the picture is completely changed when, as shown in Figs. 36 and 37, rigid connexion is used in the zone of positive moment and appropriately-graduated elastic connexion employed over the support. It is particularly apt to mention here that in the zone of negative moment in the flood approaches to the Rhine bridge at Schierstein, *Neoprene* strips were stuck to the top flanges of the steel girders before the slab was concreted, the strips acting like springs.

When considering the use of elastic connexion one must investigate whether the proposed constructional devices are economical or whether it would not be more appropriate to use some of the other methods of reducing the negative support moments which are briefly mentioned later.

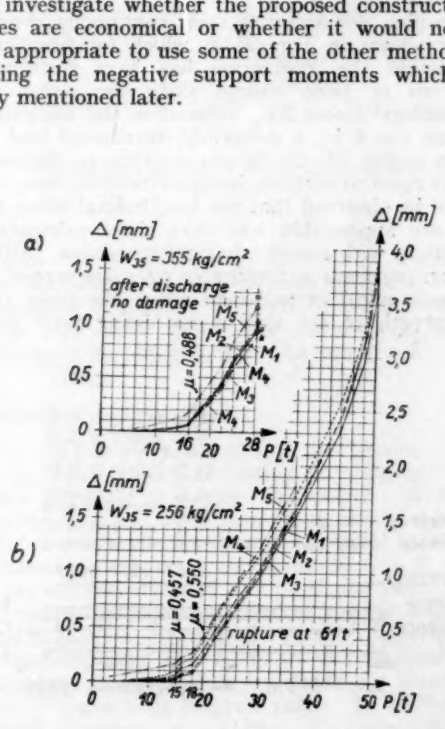


Fig. 38.—Load-displacement diagrams for push-out tests.

10. High Strength Friction Grip Bolts

The range of application of composite construction has been extended by the use of high strength bolts. The results of exhaustive tests have now been published[10], the primary object of which was to establish the permissible coefficient of friction μ between the concrete and steel. Push-out tests were performed, for example, on two rolled steel channels placed back-to-back on either side of a concrete block to which they were held by two $\frac{5}{8}$ in. diam. high strength bolts. The typical test results in Fig. 38b demonstrate that initial slip occurred at μ values varying between 0·457 and 0·550, prior to which the longitudinal strain was purely elastic. Furthermore, it will be noted that there were large reserves of strength before final collapse. The test in Fig. 40a was halted at 1·75 times the useful load, after which no distortion or damage could be observed in the concrete, channels or bolts. It is a prerequisite, however, that the channels must bear firmly on the concrete—these were laid on green concrete—and that the concrete around the holes be reinforced with simple spiral bars.

Even more favourable results were obtained with composite beams, an example of which is shown in Fig. 39. The graph of the central deflection was purely linear to a load of 20 tons and only after this load had been reached did plastic deformation take place, corresponding with a value of μ of 0·78. It should be noted that bond and shrinkage work in a favourable sense here. To obtain such a large coefficient of friction it is once again a condition that the concrete slab be firmly bedded on the steel beam and that spiral bars be provided around the holes in the slab. If the slab is not efficiently bedded—as was the case in some specially arranged tests—then the first slip will occur almost as soon as load is applied. With precast concrete slabs some suitable filler must be placed between the slab and steel flange. If this is ensured, it is possible—as tests have shown—to base calculations on large friction factors.

From the foregoing tests it may be concluded that if the above points are observed, then a coefficient of friction μ of 0·45 for the working load will be on the safe side.

Fig. 39.—Testing a composite beam with high-strength bolts as connectors.

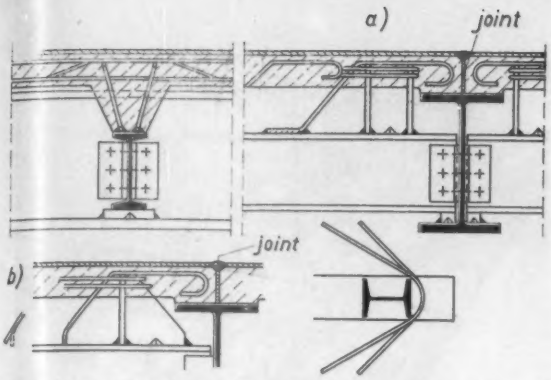


Fig. 40.—Typical details of anchor bars and end shrinkage dowels for structures.

B. PRACTICAL APPLICATIONS

11. Shear Connectors

Connectors not only provide shear connexion between the concrete slab and the steel beam and absorb the diagonal tension but, in addition, they also prevent the slab from lifting from the beam.

In strucfures it is usual to employ round steel anchor bars which can be either bent and fillet-welded to the steel beams, as in Fig. 40a, or butt-welded by some kind of stud-welding equipment, as in Fig. 40b. In the latter case, the end of the bar is initially upright, being bent over to the desired slope after welding. The rigid dowels provided at the ends of the beams to absorb the shrinkage forces normally consist of either rolled steel joists (a) or joist cuttings (b). In the middle portion of the beams where partial loading can result in the reversal of shear stresses, the anchor bars slope in both directions, as shown in Fig. 41. The broad flange beams employed were IPB 280, weighing 113 kg/m and spanning 15·6 m at 2·6 m centres. The live load was 1 t/sq.m.

The shear forces at any point in the beam can be calculated in a simple manner, as mentioned in section 8. For simply-supported beams in structures, the author has suggested the following simplification of design, which is on the safe side. If, in accordance with section 4, $N_{st,0}$ is the longitudinal force induced at the centroid of the steel beam at mid-span under full loading (dead + live load) but without creep and shrinkage, then the sum of the forces in the connectors in the outer-thirds of the beam is $0{\cdot}8N_{st,0}$ and in the middle-third $0{\cdot}2N_{st,0}$ in both directions (to allow for reversal of stress), as shown in Fig. 41. This method of calculation is valid for unsymmetrical steel beams, as well as for both mild and high tensile steel (St 37 and St 52). When compared with the requirements of DIN 1078, appreciable savings in connector steel can be made.

The force induced in the rigid end connectors can be calculated from the formula :—

$$N_E = \frac{\varepsilon_s}{\varphi_n} E_e \cdot F_{b,r} (1 - e^{-\alpha_{st}\varphi_n})$$

In the first composite floor built for the BASF at Ludwigshafen spiral reinforcement was used for shear connexion. During the 23 years until the floor was demolished it carried vibrating machinery without any damage whatsoever. Spirals take shear in both directions and also prevent the slab from lifting.

Usually, in bridge construction rectangular dowels with sloping stirrups are used. In the zones subject to reversal of shear stress the stirrups are fixed upright, as in Fig. 42. The method of calculation for such dowels is given in DIN 1078.

Fig. 41.—Anchors in industrial buildings for BASF, Ludwigshafen.

Fig. 42.—Dowels with upright stirrups.

The rigid dowels in Fig. 43 comprise stiffened angles through which anchor bars are fitted.

When there are large shrinkage forces at the ends of a bridge, dowels alone will not suffice. In Fig. 44, feather-shaped reinforcement has been used to supplement the dowels.

In America, Nelson stud connectors are used, the heads of the studs preventing the concrete from lifting.

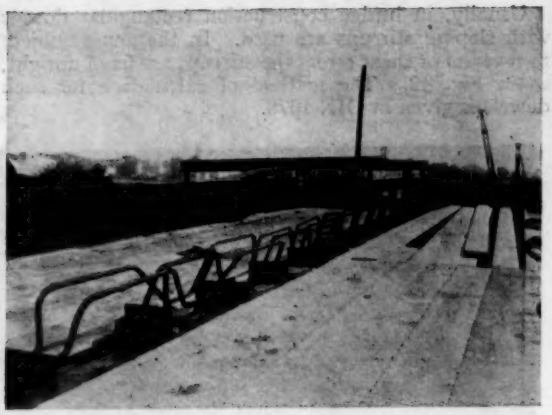


Fig. 43.—Connectors for the Cologne-Mulheim suspension bridge.

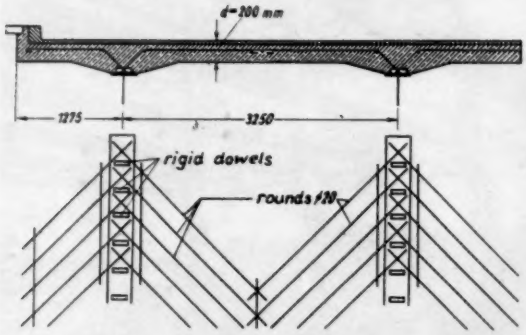


Fig. 44.—Shrinkage reinforcement at end of bridge.

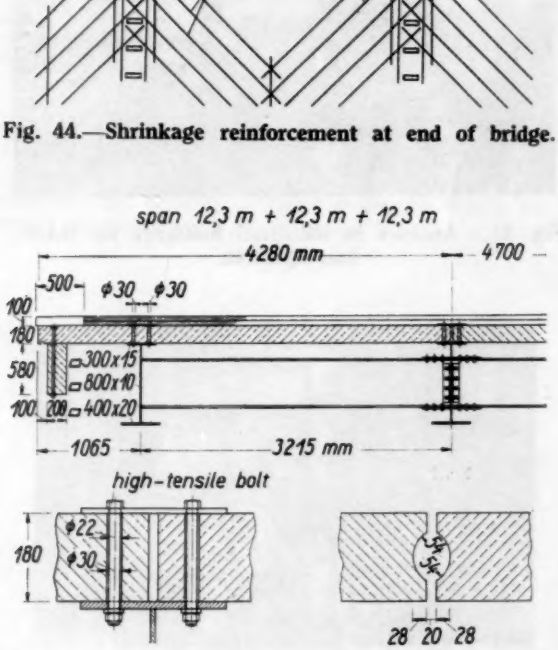


Fig. 45.—High strength bolts used with precast concrete for platform roof.

In the James River Bridge in Richmond, 235,000 studs were used. They can absorb shear from any direction and are extraordinarily easy to install.

In Fig. 45, high strength friction grip bolts have been used to connect precast concrete slabs to steel beams. They can also be used advantageously with in-situ concrete.

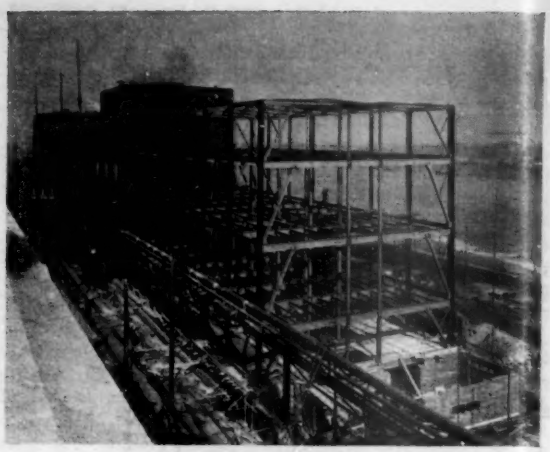


Fig. 46.—Industrial building for BASF, Ludwigshafen.

12. Structures

In structures such as shown in Fig. 46, comprising steel beams and concrete slabs, composite construction results in considerable economic advantages. It is very useful that such a steel skeleton can be erected quickly, and that the subsequently concreted floors constitute rigid slabs to conduct the forces on the walls into the vertical framing. The latter can be of portal or latticed construction or solid, as in staircases. In industrial buildings it is also reassuring to know that in later years it will be possible to carry out alterations without undue difficulty and to support services from the steel beam. The longer the span and the heavier the load, the greater are the economies which can be made. As a rule, simply-supported beams are used for simplicity of construction, thus disposing of the need for prestressing, etc. To avoid cracking, contraction joints must be provided in the slabs at the points of support of the steel beams, as shown in Figs. 40 and 48. Fig. 47 shows cracks, parallel to the steel beams in a composite deck 31 m long. *Zendler* of the BASF deduced that the shrinkage stresses resulted from the fact that the reinforced concrete deck was carried not by a rigid but by a flexible steel support, in consequence of which positive moments could occur over the

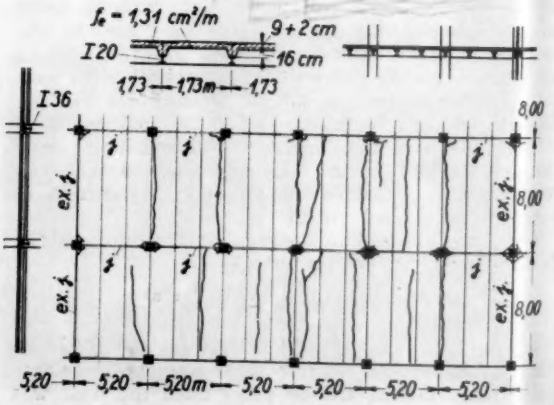


Fig. 47.—Shrinkage cracks in composite deck.

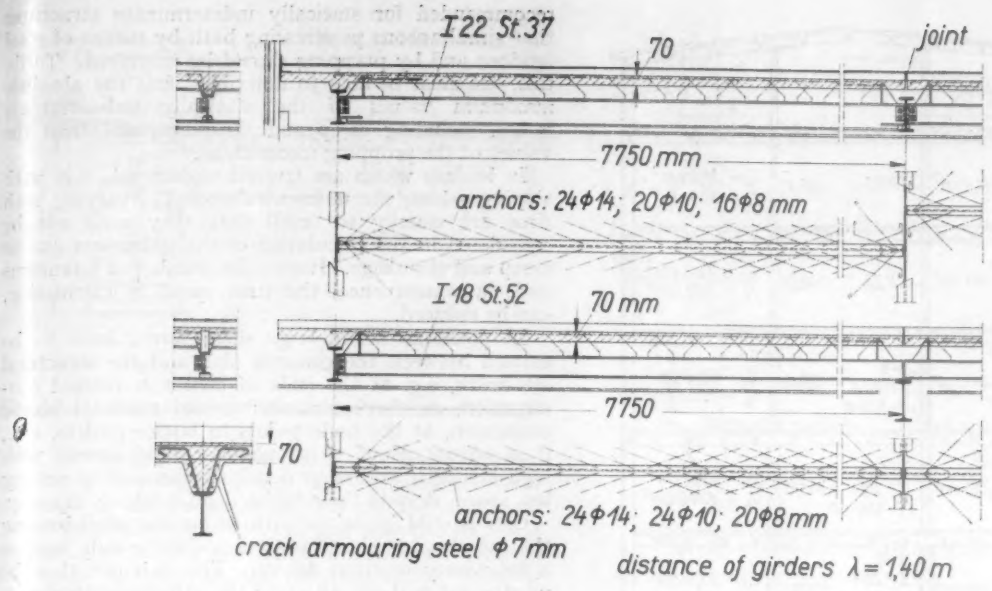


Fig. 48.—Typical details of composite floors in the Thyssen Hochhaus.

support. He suggested that in every case the bottom reinforcement should be carried right through at the supports. As will be observed, in the structure mentioned only top reinforcement was used.

The design for the composite floors of the 25-storey Thyssen skyscraper is shown in Fig. 48. High tensile steel, St 52, was used for most of the steel beams, while the slab was only 7 cm thick. DIN 4102, *Fire-resistance of structural elements*, states that such a slab should have a minimum thickness of 10 cm, but a fire test demonstrated that the proposed construction gave adequate resistance. In explanation, it has been suggested that the haunches over the steel beams produced a favourable arch effect.

Some basic rules will now be given for the design of composite decks, some of which are lifted directly from DIN 4239.

In calculating the geometrical properties of the composite section, the cross-sectional area of the concrete haunch is usually neglected as its effect on the edge stresses is very small and it can be taken as a cracked tension zone. In the composite beams for the building shown in Fig. 46, for example, it was calculated that the stresses with and without the haunch, which was 20 cm deep, were respectively 56·6 and 55·7 kg/sq.cm in the top face of the slab and respectively 1·39 and 1·41 t/sq.cm on the underside of the steel beam.

In the haunches only light crack reinforcement is used, for example, as shown in Fig. 48.

There is no reason why the depth of the haunch should not exceed 1·5 d, where *d* is the thickness of the slab, if the headroom so allows. If the deflection and construction depth are restricted, it is recommended that broad flange beams be used.

For large spans, it is economical to employ beams in high tensile steel, St 52. Using the permissible stresses for mild steel in DIN 4239, the steel beam required for a certain composite girder 12 m in span, with a slab 3 m wide and 14 cm deep and with a haunch 21 cm deep, was an IPB 260, 26 cm deep. For the same conditions a high tensile steel beam, IPB 200, only 20 cm deep, was required, saving 46 per cent in weight. If a composite section had not been employed, the alternative mild steel and high tensile steel beams would have been IPB 600 and IPB 475, respectively 60 cm and 47·5 cm deep.

Prestressing by cambering is economical as the lower edge of the steel can acquire a reserve of about 0·30 t/sq.cm if an edge stress of about 0·5 t/sq.cm is induced during the actual propping. Further propping is not practicable. Props are usually provided at the third or quarter-points. See Fig. 49.

For large spans it is economical to weld an extra plate on the bottom flange of the steel beam.

The connectors should be designed in accordance with section 11. DIN 4239 usually requires a greater, but an unnecessarily greater, cross-sectional area of steel. For the middle of a certain composite beam the area calculated and actually used was 61 sq.cm whereas that required by DIN 4239 was 118 sq.cm.

Fig. 49.—Propped cambered beams, 16 m span.

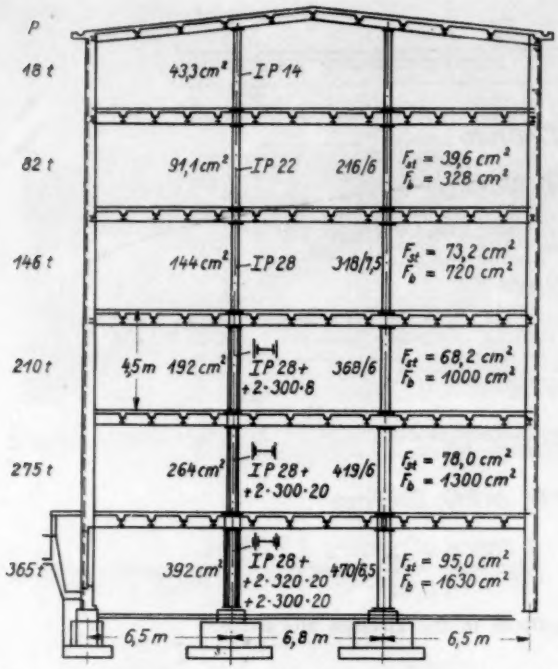


Fig. 50.—Comparison of rolled steel sections and concrete-filled tubes as stanchions.

A brief reference will now be made to composite stanchions. Fig. 50 shows a six-storey industrial building in which a comparison was made between rolled steel stanchions and tubes filled with high quality vibrated concrete. The cross-sectional areas of the tubes F_{st} and of the concrete F_b may be readily compared with those of the rolled steel columns. Arrangements were made to insert the concrete through the aperture shown in the detail in Fig. 51.

New forms of composite construction are continually being developed. An interesting example is provided by the recently-completed station roof at Munich in which 70 m span hollow triangular girders were used, the top boom of which comprised thin precast concrete slabs[11].

Acknowledgements are made to Fried. Krupp of Rheinhausen and to Herrn Regierungsbaumeister Zendler of BASF, Ludwigshafen, for providing details of the examples quoted.

13. Bridge Construction

Although widely varying forms of construction are to be described here, they have certain points in common. For instance, practically no transverse joints are provided along the length of the bridge itself. To this end a number of devices are employed, either alone or in combination, such as longitudinal prestressing of the carriageway slab, jacking of the supports, special methods of concreting or erection and elastic connexion in the zone of negative moment. As already mentioned in section 4, asphalt surfacings and isolation offer special advantages. Even in wide bridges the effective width of slab allowed by DIN 1078 is usually the greatest part of the carriageway.

In contrast with reinforced or prestressed concrete construction in which the effect of cambering is almost entirely lost through creep, this is not the case with composite construction embodying steel beams. It is recommended for statically indeterminate structures that simultaneous prestressing both by means of steel tendons and by propping should be employed. To do this, one must be able to determine first the absolute maximum values of the statically indeterminate forces, including creep and shrinkage, and then the values of the propping forces alone.

In bridges which are trussed underneath, e.g. with sloping cables, the unknown forces $X_{1,t}$ varying with time are usually so small that they need not be considered in the calculation of the unknowns due to creep and shrinkage. Hence, the number of unknowns and, in consequence, the time spent in calculation, can be reduced.

In positions where large shear forces have to be carried between the concrete slab and the structural steelwork, e.g. at the ends of cables in trussed construction, at the commencement of zones of elastic connexion, at the node points in lattice girders, etc., it is advantageous to supplement rigid dowels with high strength friction-grip bolts because they occupy less space, despite their higher load-carrying capacity.

Care should be taken with transverse steel bracing that no chord is placed near the concrete slab, save as a temporary erection device. The slab can then be prestressed without affecting the structural steelwork and, in addition, creep and shrinkage can take place without inducing stresses in the transverse direction.

The cables used in trussed construction are greased before being prestressed. In the autobahn bridge at Montabaur, in which the cables were laid in channels welded to the web plates, it was established that the coefficient of friction μ was 0·15[7]. In prestressing the cables of the flood approach spans of the North Bridge, in Düsseldorf, in which the cables change direction in a few individual points only, it was ascertained that the coefficient of friction μ over the skids on the saddles, which were treated with *Molykote*, was 0·05. In the slab in the same bridge, which was prestressed with *Leoba* tendons with an unintentional waviness of 1° per m, it was found that the value of μ was 0·15 in the transverse direction and that it varied from 0·15 to 0·20 in the longitudinal direction[12]. Depending on the site conditions, it is possible, as in prestressed concrete construction, to stress the steel as it protrudes from the slab or from a plate or recess.

Composite construction with more than two main girders is normally carried out on a grillage basis. The influence lines for the main girders can be calculated

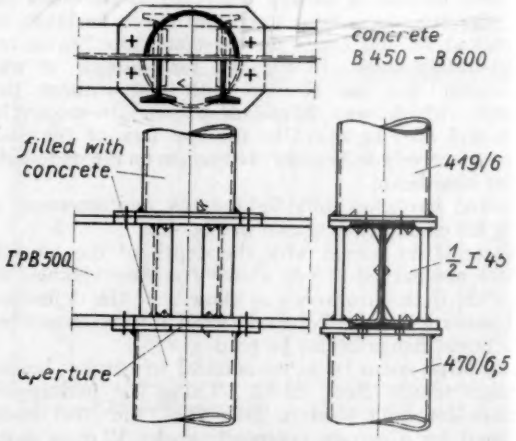


Fig. 51.—Details of joint in tubular stanchions.

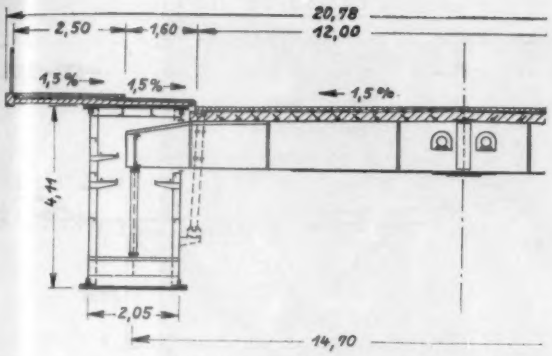


Fig. 52.—Cross-section of Koblenz-Pfaffendorf bridge.

both for statically determinate and indeterminate conditions by using either the Guyon-Massonnet method[13] or the extension of this method developed by the present author [13,14]. In this way the edge and interior girders can be made of different section. The main girders can also be torsionally weak or strong, as desired.

The following types of construction are worthy of note.

Composite cross-beams

In the Rhine bridge between Koblenz and Pfaffendorf, shown in partial cross-section in Fig. 52, only the cross-beams, which are at 3 m centres, are in composite construction. In this classical example, they are supported by props inside the main girders

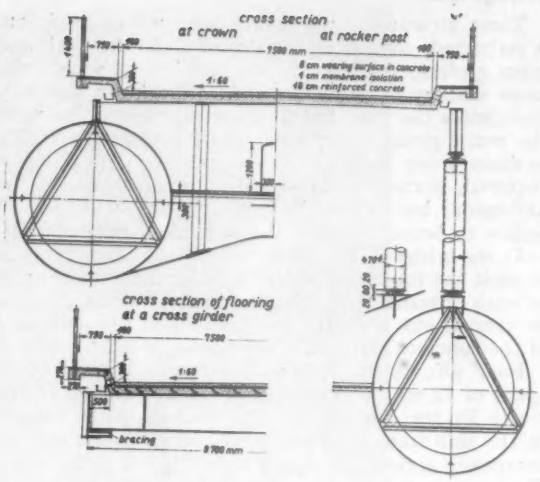


Fig. 53.—Details of Tjörn bridge, Sweden.

so that the slab can change its length under temperature, shrinkage and creep without inducing stresses in the two torsionally stiff main girders. On account of composite action the slab is subjected to compressive forces only. It is not prestressed in the longitudinal direction of the bridge.

Similarly, in the Tjörn bridge in Sweden, which comprises twin battened tubular arched ribs, 278 m in span, only the cross-beams are in composite construction, as shown in Fig. 53.

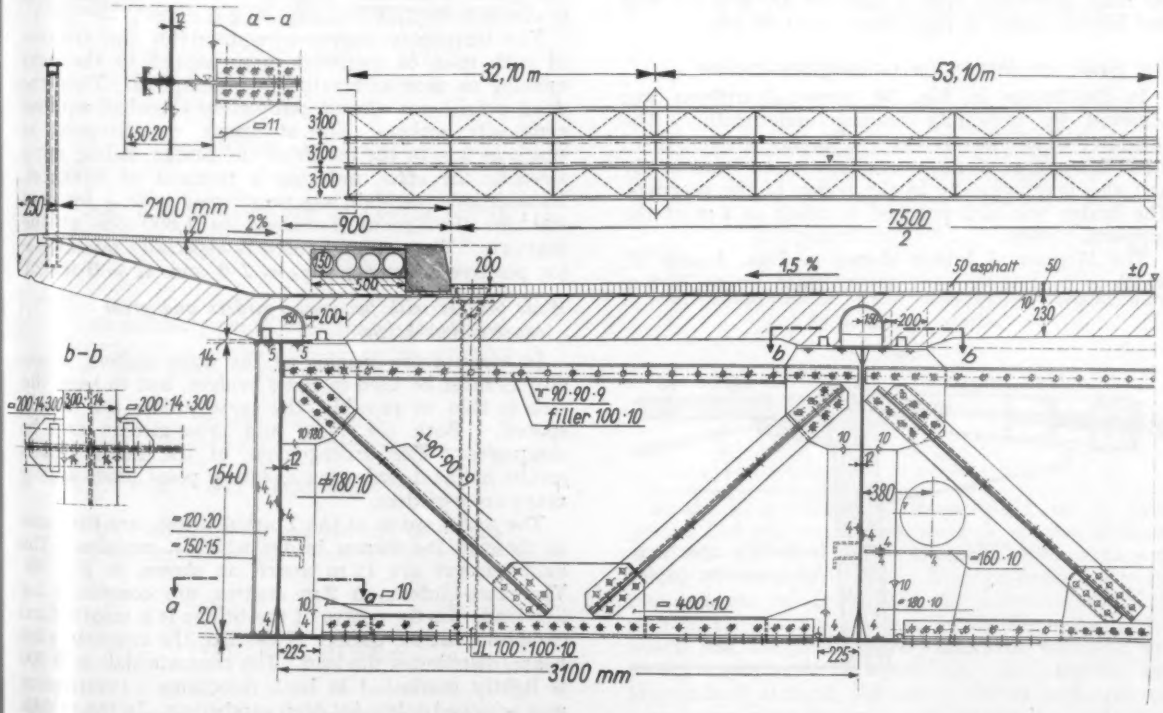


Fig. 54.—Cross-section of Ruhr bridge, Wetter, built in 1952.

Grillage decks

These structures, which are used where headroom is restricted, consist essentially of a concrete slab and main girders, the greater the number of which, the more shallow the concrete slab. The latter not only distributes the load but also acts as the top boom of the main girder. Separate cross-bracing also assists in distributing the load. If an especially rigid bridge is required, horizontal bracing can be arranged on the underside, but where bridges are very wide it will suffice to brace between the individual main girders.

In the bridge in Fig. 54 it was considered appropriate to omit the top chord of the cross bracing. Sometimes in such bracing it is recommended that the riveting be carried out after the greater part of the shrinkage of the concrete slab has taken place.

Four girders, at 5·74 m centres, continuous over spans of 72 m, are employed in the approaches to the North Bridge, Düsseldorf. To avoid excessive stresses in the slab from dead and live loading with complete composite action, the spans were trussed with cables. The sixteen cables, four per girder, which were each 63 mm in diameter and tensioned with a force of 153 tons, were anchored directly into the concrete slab at each end, as shown in Fig. 55. The slab itself was prestressed both longitudinally and transversely, thus reducing the tensile stresses in the slab to such an extent that there were no cracks and ensuring that the slab could work as the top chord of the main girder. It was more economical to tension the steel after composite action had taken place, i.e. when the concrete slab was rigidly connected to the steel girders. It was assumed in design that the concrete had the following properties: B 450 i.e. having a strength of 6,400 lb. per sq. in. at 28 days, $\varphi_n = 3 \cdot 0$ and $\varepsilon_s = 0 \cdot 00015$. By using an asphalt carpet it was possible to allow a maximum tensile stress of 30 kg/sq.cm in the top face of the concrete. The top flange of the main girder was in mild steel (St 37) and the web and bottom flange in high tensile steel (St 52).

Box girders or other torsion-resisting construction

In the bridge in Fig. 56, torsional stiffness was achieved by providing bracing underneath. The concrete, which was B 300, i.e. having a strength of 4,267 lb/sq. in. at 28 days, was prestressed transversely and also longitudinally in the region of the supports. The bridge was also propped as much as 1 m at the supports.

The Wuppertal bridge shown in Figs. 1 and 57, which is continuous over seven spans up to 72·8 m

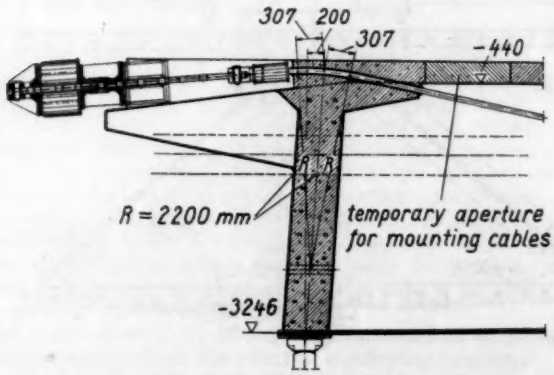


Fig. 55.—Tensioning arrangements for cables in North Bridge, Düsseldorf.

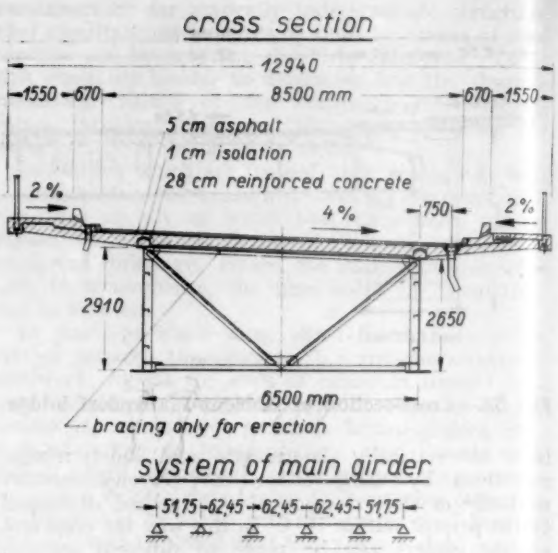


Fig. 56.—Nesselwang Valley bridge.

in length, comprises a completely closed trapezoidal section, the section being chosen because the piers were up to 40 m high. The superstructure alone would have been more economical if built with four upright girders of conventional design, the relative weights being 950 and 1,500 tons. Nevertheless, by using narrow piers about one million marks were saved. The carriageway slab, which is built in B 450 concrete is prestressed transversely with a force of 100 tons per m. The carpet consists of 5 cm of asphalt on isolation. Every 6 m along its length the box section has latticed torsion bracing. Prior to concreting it also had erection bracing along the top.

The temporary supports required for the erection of each span of steelwork were moved to the next opening as soon as riveting was complete. Thus the dead weight was almost completely absorbed without composite action. The steelwork was propped as much as 2 m in the centre of the bridge, fading away towards the ends, inducing a moment of 5,000 tm. In addition, the slab was prestressed with a force of 400 tons throughout, increased to 1,800 tons at the supports. The section chosen is especially favourable for positive moments. $\varphi_n = 2 \cdot 0$ and $\varepsilon_s = 0 \cdot 00020$.

Wide bridges with two main girders and great construction height

In view of the spacing of the main girders, cross-girders must be used in these bridges, but to keep the slab as thin as possible, the cross-girders are closely spaced. Both the main and cross-girders can be composite. The arrangement of the main girders results in the distribution of heavy point loads among many cross-girders.

The seven spans of the Loithal bridge are the same as those in the former bridge which it replaces. The main girders are 12 m apart, as shown in Fig. 58. The cross-girders, at 2 m centres, are composite for live load. In the centre of the bridge is a longitudinal composite lattice girder. This, like the concrete edge beam, distributes the load. The concrete slab in B 300 is lightly reinforced in both directions. Prestressing was achieved solely by deep cambering. In the middle of the bridge the supports were lowered as much as 3·618 m after concreting, as shown in Fig. 59.

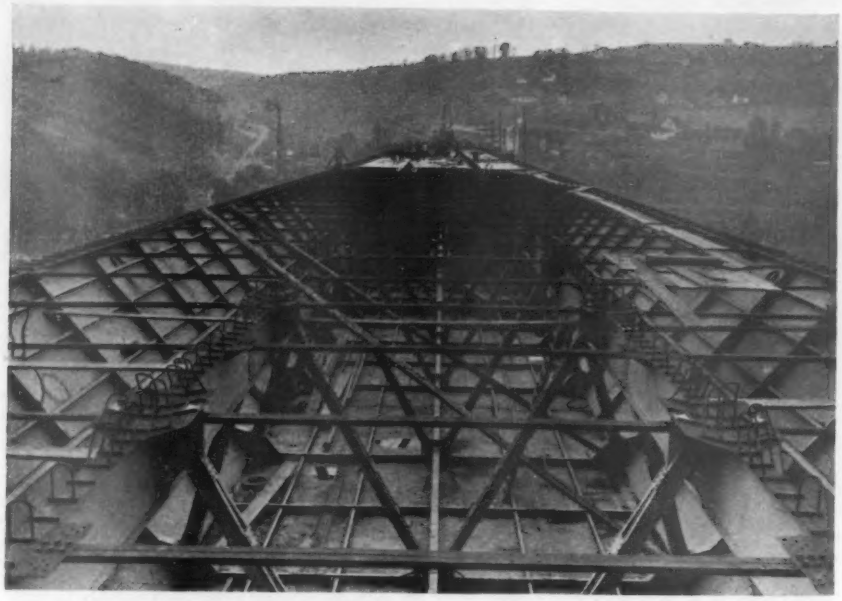


Fig. 57.—View of Wuppertal-Oehde Bridge during construction.

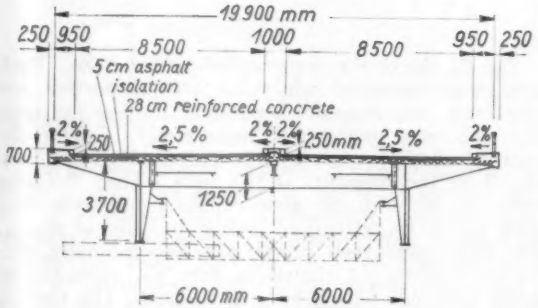


Fig. 58.—Cross-section of Loithal Bridge.

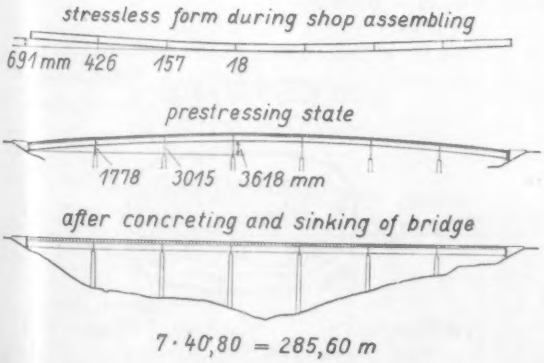

stressless form during shop assembling
691mm 426 157 18
prestressing state
1778 3015 3618 mm
after concreting and sinking of bridge
7 · 40,80 = 285,60 m

Fig. 59.—Prestressing arrangements for Loithal Bridge.

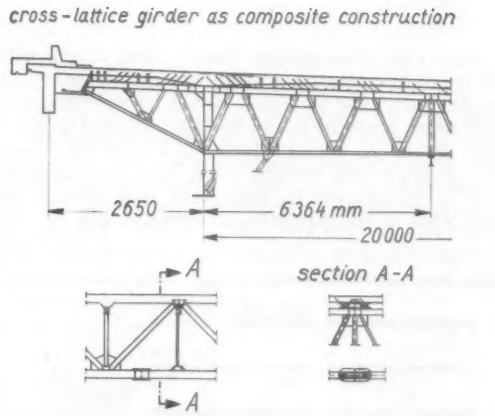


Fig. 60.—Details of Kauppen Bridge.

In the Kauppen bridge shown in Fig. 60 the main girders had the extraordinarily wide spacing of 20 m, as a consequence of which latticed cross girders were employed at 2 m centres. In accordance with the requirements of DIN 1078, the effective width of slab for the main girders was 14 m. The cross-girders which had a special chord under the concrete slab were composite for live loading. In addition, two longitudinal latticed composite girders and concrete edge beams were designed to distribute the loads.

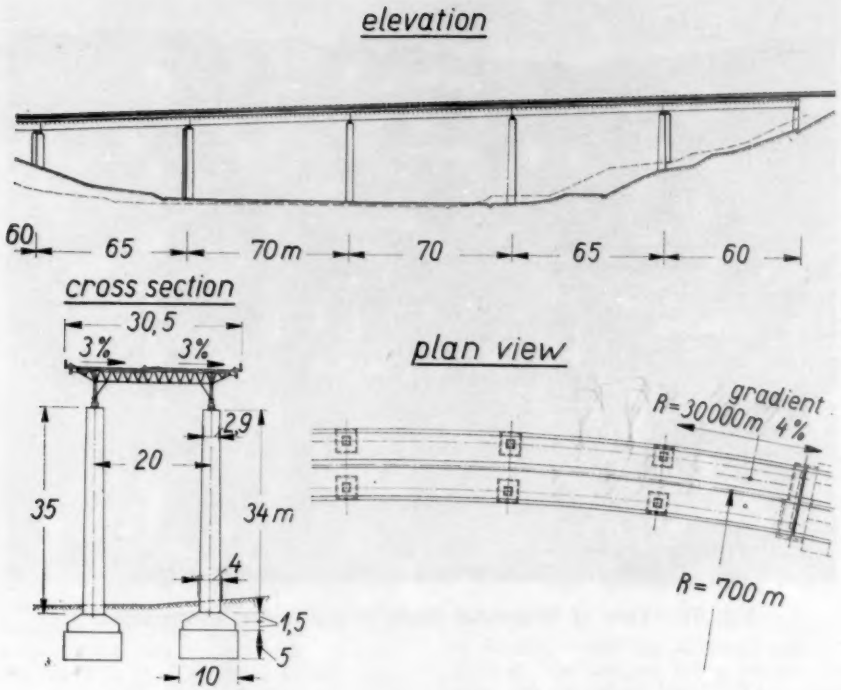


Fig. 61.—The Kauppen Bridge.

Had the cross-girders been constructed without the special top chord the creep and shrinkage conditions in the transverse direction would have been considerably more favourable and 300 tons of steel would have been saved. As it was, additional transverse tensioning with prestressing tendons must have been required.

It is interesting to note that individual columns were used at the supports, as shown in Fig. 61.

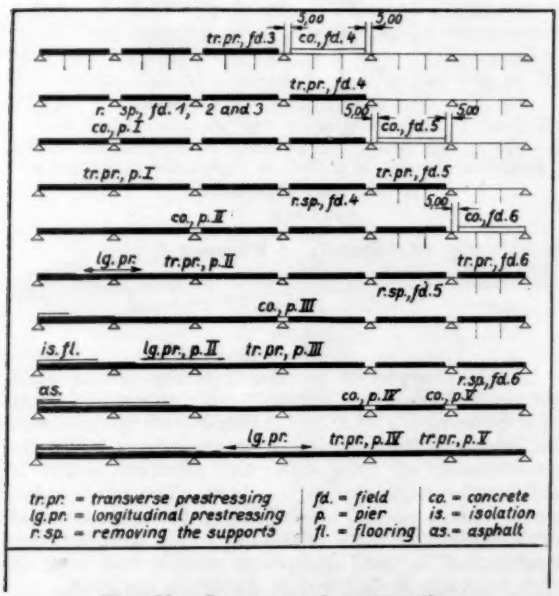


Fig. 62.—Sequence of construction.

Fig. 62 shows the sequence of construction. Each span was supported while the central portion was concreted, the supports remaining in position until the slab was prestressed transversely. After the removal of the supports, only the steel girders were absorbing the moments at the piers. Then the slabs were concreted in the zone of the piers, their dead load again being carried solely by the steelwork. The slabs were then prestressed longitudinally with Leoba tendons, as shown in Fig. 63, each tendon being tensioned with a force of 33 tons. The tendons are shown in position in Fig. 64. Near the piers some of the tendons were laid obliquely so that the heavy tensile loads there could be conducted into the main girders. In fact, the slab, which was in B 450, was tensioned longitudinally throughout its whole length. The following factors were used in calculation: $\varphi_n = 2 \cdot 5$, $\varepsilon_s = 0 \cdot 00020$.

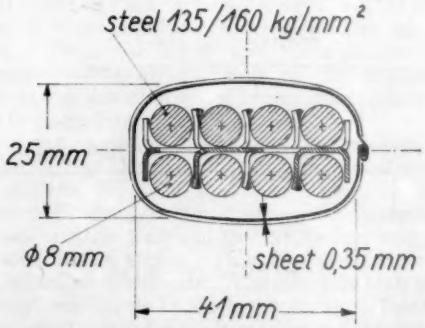


Fig. 63.—Leoba tendon.

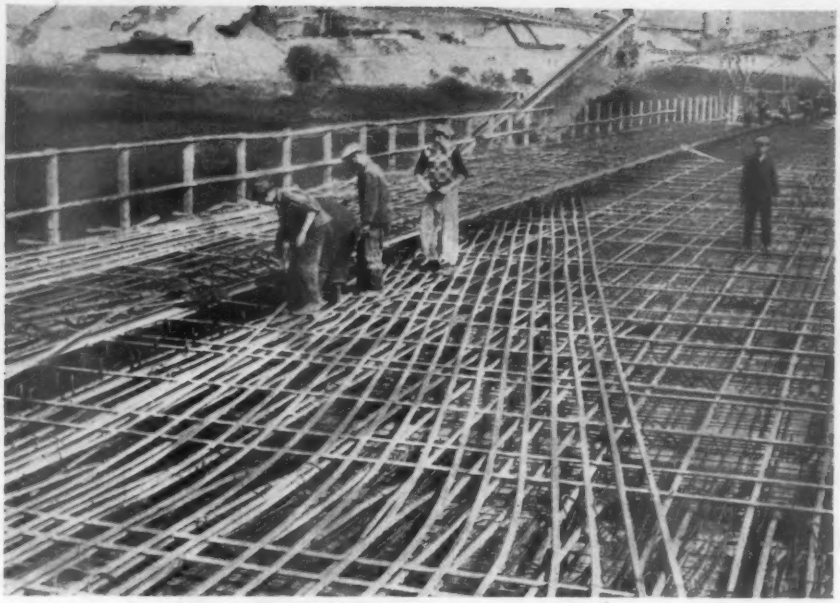


Fig. 64.—Arrangement of tendons near the piers.

Bowstring bridges

Figs. 65 and 66 show details of a Langer (bowstring) bridge designed by *Dornen*, in which the longitudinal and cross-girders are in composite construction. Initially the steelwork was erected with a span of 46·8 m, the joints between the arch ribs and ties being left unwelded. The tie was then tensioned with a force R of 47 tons, which cambered it and reduced the span to about 46·7 m, as shown in Fig. 66a. After concreting the cable carried a force P_v of 137 tons, but when this force was released, the deck sank and the tie resumed its original length of 46·8 m, as shown in Fig. 66b, thus prestressing the composite section by compressing the slab. After site welding the joints between each arch and tie, the cable, which is situated at the centroid of the composite section, was tensioned with a force P_v of 306 tons.

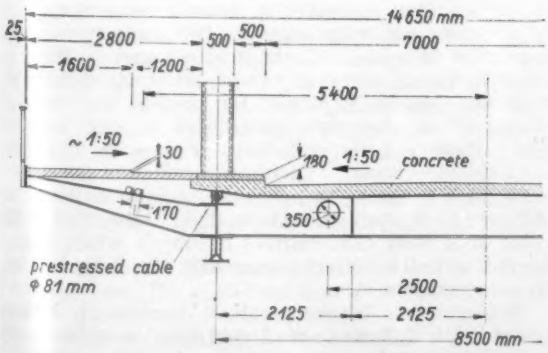


Fig. 65.—Gahmenerstrasse Bridge.

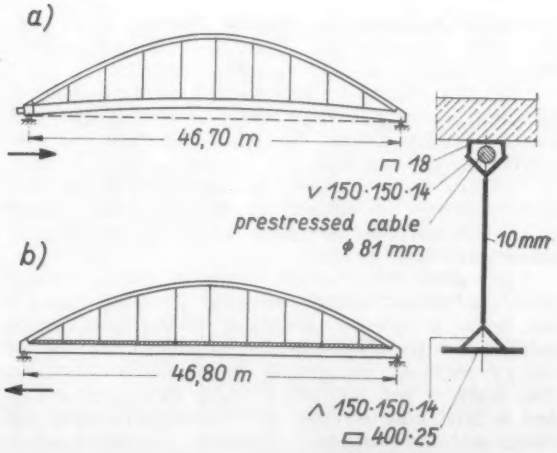


Fig. 66.—Erection conditions.

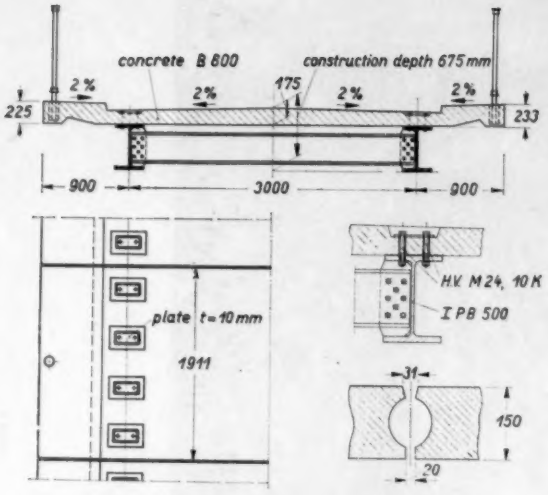


Fig. 67.—Cross-section of Ems footbridge.

Bridges with high strength bolts for shear connexion

The use of high strength bolts with precast concrete slabs has already been mentioned. Fig. 67 shows an example in which this mode of construction was employed for a footbridge over the Ems. Special care is needed in making the joints between the slabs but the remaining work is simple.

Further information on composite bridges is given in Reference 16.

Acknowledgement is made to the following who have kindly furnished information: Herrn Ministerialrat Dr.-Ing. Klingenberg and Dr.-Ing. Grassl, as well as the firms Demag, Dörnen, Krupp, MAN and Neusser Eisenbau.

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11. Hoffman, E., and Latzin, K. "Zur Berechnung und Konstruktion der neuen Bahnsteighalle München Hbf." *Bauingenieur* 35 (1960), p. 207.
12. Sattler, K. Nordbrücke Düsseldorf 1958. Herausgegeben von der Landeshauptstadt Düsseldorf im Springer-Verlag.
13. Sattler, K. "Betrachtungen zum Berechnungsverfahren von Guyon-Massonnet für frei aufliegende Trägerroste und Erweiterung dieses Verfahrens auf beliebige Systeme." *Bauingenieur* 30 (1955), p. 77.
14. Sattler, K. "Betrachtungen über Trägerroste mit Steifigkeitsunterschieden zwischen Rand-und Innenträgern." *Bauingenieur* 34 (1959), p. 1.
15. Dörnen, K., and Meyer, A. "Die Emsbrücke Hembergen in dübellosem Stahlverbund." *Der Stahlbau* 29 (1960), p. 199.
16. Klingenberg, W. "Brückenbauten an Bundesfernstrassen." *Bauingenieur* 32 (1957), p. 245.

Book Review

Energy Principles in Applied Statics, by T. M. Charlton. (London and Glasgow: Blackie, 1959). 9¼ in. × 6¼ in., 112 + vii pp. 25s.

This slim book deals with the fundamentals of potential energy, virtual work, strain energy and complementary energy methods of analysis. Taken merely as an extension of a conventional training it will give many engineers the feeling that they are understanding the subject for the first time. (they may even feel that the book would be revealing to many a textbook writer)

Apart from the primary virtue of dealing clearly with the subject matter there is, dispersed through the book, a valuable discussion of the most useful method for particular structures and of the use of energy methods in getting approximate solutions. The work is not limited to lineal structures and it has a brief note on the use of energy methods for elastic stability problems. It ends with an introduction to stiffness and flexibility coefficients. Particularly welcome is the wide outlook; the author's concern is to discuss energy methods, he is quite ready to conclude that they are not much use if that is where he is led.

In order to avoid distracting complexity the examples are all very simple, this backfires a little; it becomes trying at times to follow solutions stated at length when one can solve almost by inspection from resolution of forces. There is also a certain flatness of style giving every statement an equal weight so that one has to highlight for oneself the turning points of an argument (possibly a greater wisdom this; there is little uniformity perhaps in what we consider convincing). Castigliano comes in for something of a battering, which seems a little unfair: he did, after all, start the whole thing. But this is to cavil at a book that clarifies a subject which badly needs it as well as acting as something of an introduction to many modern ideas.

Presentation (between rather unpleasant brown covers with difficult-to-read gold title on spine only) is excellent—clear type in ample space, figures, mathematics and text well related and all properly numbered for reference. S. W.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, at 5.55 p.m. on Thursday, 23rd February, 1961. Lt.-Colonel G. W. Kirkland, M.B.E. (Mil.), M.I.Struct.E., M.I.C.E., (President) in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

STUDENTS

BASTICK, Ronald William, of Hatch End, Middlesex.
BRAND, Terence David, of Harrow, Middlesex.
DEWAR, Barry Robert, of Caterham, Surrey.
DRAYTON, Roger Evan Thomas, of Uxbridge, Middlesex.
FRANCIS, Warren, of Bulawayo, Southern Rhodesia.
GRANT, Iain Lockhart, of Kenya, East Africa.
JOHNSON, David, of West Gorton, Manchester.
KARUNANANTHAM, Punniamoorthy, of Chenkaladi, Ceylon.
MORRISON, John, of Bracknell, Berkshire.
POLLOCK, Ian George, of Salisbury, Southern Rhodesia.
REGIS, Nicholapillai Joseph Anton, of Colombo, Ceylon.
SAUNDERS, James Robert, of Stapleford, Notts.
STANBRIDGE, Graham John, of Burton upon Trent, Staffs.
TANG AH TEE, of Brighton, Sussex.
YOONG, Ernest, of Perak, Malaya.

GRADUATES

BANSI, Purushottam Gobindram, B.E., of Bombay, India.
BARRATT, Gordon, of Cheadle, Cheshire.
BEEVERS, Clifford, of Swinton, Nr. Mexborough, Yorks.
BUXTON, Raymond, of Clay Cross, Nr. Chesterfield, Derbyshire.
CHAWLA, Manmohan Singh, of Punjab, India.
CRITCHLEY, Norman Andrew, of Bolton, Lancs.
DOVE, Jeffrey, of Swanwick, Derbyshire.
FLETCHER, Robert, of Rotherham, Yorks.
FREEMAN, Charles Ernest, of Brentwood, Essex.
GILLAM, Clive Edward, of South Croydon, Surrey.
GOH TOCK ENG, B.E., of Singapore.
GRIFFITHS, Brian John, of South Ruislip, Middlesex.
HEALEY, Thomas Carl, of Smethwick, Staffs.
HOCKHAM, Alec James, of London.
IRVING, Arthur Edward, B.Sc., of Teddington, Middlesex.
JACKSON, Eric, of Toton, Nottingham.
LEE MOON HOE, B.E., of Malacca, Malaya.
LEE, Peter Bay Tseng, B.Sc., of Singapore.
MCNAIR, Samuel James, of Dunlop, Ayrshire, Scotland.
MAIN, Peter Keir, of Brisbane, Queensland, Australia.
MAKIN, William Alan, of Bury, Lancs.
MANTHE, Benny Oswald, of Johannesburg, South Africa.
MILLARD, Francis Edward, of Dagenham, Essex.
MYERS, Aubrey Norman, of Umbilo, Durban, South Africa.
NICHOLLS, Joseph Thomas, of London.
ONIONS, John, B.Sc., of Lincoln.
OYEBANJI, Aderemi Ijaola, of London.
PANDIT, Tarit Kumar, B.E., of Sheffield.
PATANKAR, Vinodkumar Dattatraya, B.E., of Bombay, India.
QURESHI, Shujaat Ullah, M.Sc., B.E., of London.
RHODES, Keith Graham, B.Sc., of Rothwell, Nr. Leeds.
SETHURAMAN, S., M.Sc., B.E., of Rourkela, Orissa State, India.
STANTON, Michael Thomas, of Solihull, Warwickshire.
STANYER, Anthony John, of London.
TAYLOR, Michael Anthony, B.Sc., of Stretford, Lancs.
TOMLINSON, Harold, of Sutton-in-Ashfield, Notts.
TOWNSEND, Gerald Hughes, of Reading, Berkshire.
WESTLAKE, Richard Piers, of Fordingbridge, Hampshire.

ASSOCIATE-MEMBERS

DHAR, Paramesh Ranjan, B.Sc., B.E., of London.
MARLEY, Stanley James, of Liverpool.
TAYLOR, Michael William, of London.
WINGATE, Gerald, of Richmond, Surrey.

MEMBERS

GOLLEDGE, Stephen Edward, A.M.I.C.E., of Caerleon, Monmouthshire.
PARKINSON, Graham Charles, of London.

TRANSFERS

Students to Graduates

FLASHMAN, Archibald Edward, of Epsom, Surrey.
GILMOUR, Peter James, of Manchester.
GUPTA, Siba Prosad, B.Sc., of Lanesfield, Nr. Wolverhampton, Staffs.
STOCKS, Keith Bernard, of London.
TOMLINSON, John Lloyd, of Darlington, Co. Durham.

Graduates to Associate-Members

BELL, Thomas John, of Airdrie, Lanarkshire.
EVERETT, Richard William, B.E., of Cooma, N.S.W., Australia.
NARAYANAN, Ramasubba, B.E., M.Sc.(Eng.), of London.
SHERBOURNE, Archibald Norbert, B.Sc., of Cambridge.
THORPE, Edwin Holt, of Ashby, Scunthorpe.

Associate-Members to Members

BANNISTER, John Lloyd, M.Sc., A.M.I.Mun.E. of Cardiff.
CHAMPION, Stewart, Ph.D., M.Sc., A.M.I.C.E., of Coulsdon, Surrey.
CUTTER, Joseph, of London.
HUGHES, Alfred Charles, M.Eng., A.M.I.C.E., of Bristol.
NEEDHAM, Frederick Harold, B.Sc., of West Wickham, Kent.
NICHOLSON, Thomas Hedley, B.Sc., A.M.I.C.E., of Ponteland, Northumberland.
TIPP, Eric James, of Petts Wood, Kent.
WONG, Albert Peng Choon, B.Sc., of Singapore.

Associate-Members to Retired Associate-Members

FROST, Robert William, of Wallington, Surrey.
LANCASTER, John Kenneth, of Haywards Heath, Sussex.
PHILLIPS, Herbert Eric, of St. Ives, Cornwall.
TELLIS, Charles Sebastian Joachim, B.A., B.Sc., A.M.I.C.E., of Poona, India.
WRIGHT, George Lisson, of Highcliffe, Hants.

Associate to Retired Associate

CURTIS, Herbert Foster, of London.

Members to Retired-Members

ABRAHAMS, Herbert Edward, of Chislehurst, Kent.
ANDERSON, John, M.I.C.E., of Cults, Aberdeenshire.

CASWELL, Fred, of Bristol.
KIRKWOOD DODDS, Archibald, M.C., F.R.I.B.A., of London.
HILEY, Bernard Arthur Ernest, M.I.C.E., of London.
NAISH, Hubert Francis, B.Sc., A.M.I.C.E., of Kingston, Nr. Lewes, Sussex.
NORRIS, Harold Wilfred, of Eastbourne, Sussex.
RICHARD, Thomas Bryn, of Sketty, Swansea, Glam.
SHIELDS, Malcolme Irvine, of Kilsyth, Glasgow.
YOUNG, James Heuchan, of Redcar, Yorkshire.

OBITUARY

The Council regret to announce the deaths of Frank ABBEY, Thomas W. CORKHILL, George Ashley HANBY, Ralph Willans MARSDEN, and Professor William Norman THOMAS (Retired Members); Reginald Palmer BAINES (Associate); Cyril George RICHARDSON (Associate-Member), Harry Nigel BRATT (Graduate).

RESIGNATIONS

Notice was given that the Council had accepted with regret the resignations of Frank Joseph BOWN, Thomas DAWE, Thomas MCINTYRE (Members); Charles Edward BEAN, Robert Peel MEARS, Major Richard Aloysius QUIGLEY (Retired Members); Hugh Henry Boyd STEWART (Associate); Charles Hugh BELL, Jamshed Ruston IRANI, Charles MAIDSTONE, Charles Vernon MILLER, Royden Hibbert NEWMAN (Associate-Members); Frederick Joseph FINDON (Retired Associate-Member); Andrew Keith Murray BROWN, Michael Henry CLARKE, John Albert CONDER, Timothy KAO WU, Colin KENDALL, Ramchandra Becharbhai PATEL, Richard Watkinson SANDERS, John Ashton TINKLER (Graduates); Leslie Bentley KNOTT, Roland Derek PATTISON (Student Members).

APPOINTMENT OF SECRETARY

The Council have appointed Mr. C. D. Morgan, F.C.I.S. to succeed Major R. F. Maitland, O.B.E., M.I.Struct.E., as Secretary of the Institution. Major Maitland, who has held office since 1930, is retiring at the end of September and Mr. Morgan will take over on the 1st October, 1961.

HONOURS AND AWARDS

In offering their sincere congratulations to the following members on the distinctions recently conferred upon them, the Council are also expressing the good wishes of the Institution.

Order of the British Empire—C.B.E.
Brigadier C. C. Parkman (Member).

Army Emergency Reserve Decoration—E.R.D.
Lt.-Colonel F. D. Ogden (Associate-Member).

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Wednesday, 12th April, 1961

An additional Ordinary Meeting, not printed in the Sessional Programme, has been arranged and will be held at 11, Upper Belgrave Street, London, S.W.1, on Wednesday, 12th April, 1961 at 6 p.m., when a paper on "Structural Engineering in Nigeria" will be given by Mr. A. Brimer, A.M.I.Struct.E., (Honorary Secretary of the Nigerian Section and the Institution's Representative in Nigeria).

Thursday, 27th April, 1961

Ordinary General Meeting for the election of members 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Professor Dr. tech. h.c., Dr.-Ing. Konrad Sattler will give a paper on "Composite Construction in Theory and Practice."

Thursday, 25th May, 1961

Ordinary General Meeting for the election of members at 5.55 p.m., followed by the Annual General Meeting at 6 p.m.

Thursday, 22nd June, 1961

Ordinary General Meeting for the election of members 5 p.m.

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS, JULY, 1961

The Examinations of the Institution will next be held in the United Kingdom and overseas on the 11th and 12th July, 1961 (Graduateship) and the 13th and 14th July, 1961 (Associate-Membership).

CHANGE OF TELEPHONE NUMBER

The telephone number of the Institution will be changed to BELgravia 4535 (3 lines) at 1.30 p.m. on the 19th April, 1961.

REPRESENTATION

The Council have appointed the following Institution representative:

British Standards Institution Thermal Transmission through Building Structures Sub-Committee SEF/14/8: Mr. A. R. Mackay (Associate-Member).

COMMONWEALTH TECHNICAL TRAINING WEEK

The Institution is staging an exhibition of photographs, models and drawings illustrating the profession of Structural Engineering from 29th May to 2nd June, at 11 Upper Belgrave Street, London S.W.1.

The exhibition will be open daily from 10 a.m. to 12 noon and from 2 p.m. to 8 p.m. Members of the Institution will be available throughout to answer questions regarding training for the profession and prospects.

During the mornings of Monday 29th May, Wednesday 31st May, and Friday 2nd June, coaches will be provided to take those interested to visit outstanding structures under construction such as Hammersmith Flyover, the Royal College of Arts, and some of the new high buildings.

During the afternoons of those days illustrated talks will be given on various aspects of Structural Engineering and on the Tuesday and Thursday there will be film shows.

It is hoped that members and their friends who are able to do so will visit the exhibition, particularly between 6 and 8 o'clock each evening.

DRURY MEDAL AWARD

The alternative subjects for this eighth Competition, are a pipe-bridge and a petrol service station.

Graduates and Students of the Institution not over 27 years of age are invited to apply for full details to the Secretary, envelope to be marked in the top left-hand corner 'Drury Medal Award.'

The general conditions of the competition are as follows:

(a) The competition shall be for Graduates and Students of not more than 27 years of age.
(b) The subjects of the competition will be designs of a structural character, that is to say, involving structural design rather than planning.

(c) The subjects of design and the conditions shall be prepared and issued biennially.

(d) A jury shall be appointed to examine the work submitted and to interview candidates if found necessary.

(e) In order to ensure that the design submitted is the unaided work of the competitor, the drawings, calculations, etc., submitted shall be endorsed by the candidate : "I declare that the work I hereby submit is my own unaided work." The declaration shall be signed by the competitor, and be either countersigned by a corporate member, or be certified as made before a Justice of the Peace, or a Commissioner for Oaths.

LAPSED MEMBERSHIP

The membership of the undernoted has lapsed in accordance with the provisions of Bye-Law 33 :

James BOWIE, Richard Goulden BRICKELL (Members) ; Harold Vivian Charles BELLMAN, Chhatumal Jhamandas CHANGULANI, George Bowen FRITCHLEY, Frederick George RIDOUT (Retired Members) ; Leonard FREEBORN, Colonel Joseph CALDWELL (Retired Associates) ; James McLachlan FAIRLIE, Francis David GOODMAN, Michael Edward HARKIN, Frank Webster HOLLOWAY, Norman HUTCHINSON, William JONES, Ihsan NAMAN, Alan TAYLOR, Ernest Charles TURNER (Associate-Members) ; Lt. Harold Norman ANDREWS, Osman el AWAD, Santosh Kumar BANDYOPADYAY, Narendra Nath BANERJEE, Wladyslaw Otton BIERNACKI, Erik Hugh BIRD, William Harold BOOTH, Fred BROOKSBANK, Thomas Edmund BURTON, Nikhil Nath CHATTERJEE, Antoney Ariyasena DE SILVA, Glyn Alan FOX, John Eltham FROST, Issac Salim GABBAY, Kaikobad Manecksha GANDHI, Albert Francis GARRITY, Kironmoy GHATAK, William James GILLBANKS, Purushottam Shankar GODBOLE, Leslie George HARMSWORTH, Robert Michael HILL, Mohed Ibrahim IBRAHIM, Neil McLeod JEFFREY, Kailash Chander KATHURIA, Anthony John KAY, Mahmood Aslam KHAN, James Edward LEWIS, John Bernard MANNING, George Keith MESSERVY, Zahee Mirza MIRZA, Makund Govind NATU, Donatus Onyejela OPARA, Fred PICKERING, Arvind Ganpatrao RAJE, Coorg Krishniengar RAMESH, Patrick Anthony RAYNER, Mamnoon Ishak SAWERIS, Baldip Singh SEHMI, Joseph Olawole SHOJOBI, Sumanlal Dipchand VORA, Raymond Edward WATKINS, Raymond Brian YOUNG (Graduates) ; Keith HARTLEY, Dennis Lionel HECKLER, George Alfred JOHNSON, Louis LEVTOV, Hugh Swan LIDDELL, Peter O'BRIEN, Michael Richard Holmes PEGLER, Nirmaljit SINGH (Students).

ADDITIONS TO THE LIBRARY

American Concrete Institution Journal, 1943-1958. Presented by Mr. G. H. Hodgson.

BILLIG, Kurt. *Structural Concrete*, London, 1960. Presented by Mr. W. Hunter Rose.

CARPENTER, S. T. *Structural Mechanics*, New York & London, 1960. Presented by Mr. L. S. Watts.

CORNICK, H. F. *Dock and Harbour Engineering, Vol. 3. Buildings and Equipment*, London, 1960.

DURELLI, A. J., PHILLIPS, E. A. and TSAO, C. H. *Introduction to the Theoretical and Experimental Analysis of Stress and Strain*, New York and London, 1958. Presented by Mr. T. C. Waters.

DUTRON, P. *Restoring Concrete Surfacings to their correct-levels by Pressure Grouting and Re-surfacing with Cement Mortar*, London. C. & C. A. Translation. No. 92, 1961.

FABER, O. *Reinforced Concrete, 2nd Ed. rewritten and extended by John Faber and Frank Mead*, London, 1960.

GRAY, C. S., KENT, L. E., MITCHELL, W. A. and GODFREY, G. B. *Steel Designers' Manual, 2nd Ed.*, London, 1960. Presented by Mr. G. Bernard Godfrey.

GUYON, Y. *Prestressed Concrete, Vol. I, 4th Impression*, London, 1960. Presented by Mr. T. C. Waters.

Institution of Civil Engineers, Journals, November 1936-*December* 1951, *Proceedings, January* 1952-*September* 1960, *and "Chartered Civil Engineer,"* 1949-1957 Presented by Mr. F. C. J. Bagnall.

KEENE, P. W. *A Preliminary Examination of the Vebe Consistometer*. London, C. & C. A. 1960, Technical Report, TRA/343.

KINNUNEN, S. and NYLANDER, H. *Punching of Concrete Slabs without Shear Reinforcement*. Stockholm, 1960.

PELTIER, R. *Prestressed Concrete Roads : Tests and Future Prospects*, London, C. & C. A. Translation. No. 91, 1961.

Smithsonian Institution. Anuual Report of the Board of Regents, 1959, Washington, 1960.

Symposium (Second) on Naval Structural Mechanics Proceedings-Plasticity, Oxford, 1960.

Symposium on Nuclear Reactor Containment Buildings and Pressure Vessels, Royal College of Science and Technology, Glasgow, 1960.

WALCH, O. *Baumaschinen und Baueinrichtungen, 3rd Ed. Vol. III Übungsbeispiele*, Berlin/Göttingen/Heidelberg, 1958. Presented by Mr. K. R. Northgreaves.

The following Paper has been added to the Library : "*The American Ballistic Missile Early Warning System*," by Leo Walter.

JOURNAL CASES AND BINDING, 1960

A binding case can be supplied for the twelve issues of the Journal, January—December, 1960 (Volume 38), price 12s. 6d., including postage. The price for binding in half leather, inclusive of packing and postage, is 31s. 0d. per volume.

It is requested that all parcels containing Journals forwarded for binding should bear the name and address of the sender, and must be dispatched to reach the Institution by the 28th April, 1961.

An index will be included in all bound volumes.

Members making their own arrangements for binding may obtain a copy of the Index upon application to the Secretary.

Branch Notices

LANCASHIRE AND CHESHIRE BRANCH

The following meeting has been arranged :—

Thursday, 13th April, 1961

Annual General Business Meeting followed by a paper "Power Stations as a Structural Problem," by Mr. H. Dickson, A.M.I.Struct.E. At the College of Science and Technology, Manchester, commencing at 6.30 p.m. Light refreshments will be available from 5.45 p.m.

MERSEYSIDE PANEL

The following meetings have been arranged :—

Thursday, 6th April, 1961

Annual General Meeting of the Merseyside Panel. At the Liverpool Engineering Society, The Temple, Dale Street, Liverpool.

Friday, 12th May, 1961

Annual Dinner Dance. At the Carlton Club, Eberle Street, Liverpool.

All meetings will commence at 6.30 p.m. and will be preceded by light refreshments at 5.30 p.m.

Joint Hon. Secretaries : Wm. S. Watts, M.I.Struct.E., A.M.I.C.E., 11, Newchurch Lane, Culcheth, Nr. Warrington, Lancs., and M. D. Woods, A.M.I.Struct.E., 8, Dennison Road, Cheadle Hulme, Cheshire.

MIDLAND COUNTIES BRANCH

The following meeting has been arranged :—

Friday, 28th April, 1961

Thirty-fifth Annual General Meeting. Mr. G. B. Smedley, B.Sc.(Tech.), A.M.I.Struct.E., A.M.I.C.E., and Mr. G. M. Mills, B.Sc.(Tech.), A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E., on " Modern Urban Highway Structures."

At the Byng Kendrick Suite, College of Advanced Technology, Gosta Green, Birmingham, 6, at 6.30 p.m. Tea will be served from 5.45 p.m.

Hon. Secretary : S. M. Cooper, M.I.Struct.E., " Applegarth," 56, Hyperion Road, Stourton, Stourbridge, Worcestershire.

GRADUATES' AND STUDENTS' SECTION

The following meeting has been arranged :—

Friday, 7th April, 1961

Address by the Branch Chairman, Major G. Botteley, R.E., T.D., M.I.Struct.E., M.I.C.E., on " The Reconstruction of Pershore Lock." Followed by the 14th Annual General Meeting.

At the Engineering Centre, Stephenson Place, Birmingham, at 6.30 p.m. Tea will be served from 6 p.m.

Hon. Secretary : H. T. Dodd, Shepherd's Cottage, Grove Lane, Wishaw, Sutton Coldfield, Warwickshire.

NORTHERN COUNTIES BRANCH

The following meetings will be held in April :—

Wednesday, 5th April, 1961

Lecture to be arranged. At Newcastle.

Thursday, 6th April, 1961

Annual General Meeting. Lecture to be arranged. At Middlesbrough.

Friday, 28th April, 1961

Annual Dinner, at the Corporation Hotel, Middlesbrough.

Unless otherwise stated meetings at Newcastle will be held at the Neville Hall and those at Middlesbrough at the Cleveland Scientific and Technical Institution, and will commence at 6.30 p.m. preceded by a buffet tea at 6 p.m.

Hon. Secretary : P. D. Newton, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., 6, Cornfield Road, Linthorpe, Middlesbrough, Yorks.

NORTHERN IRELAND BRANCH

The following meeting has been arranged :—

Monday, 10th April, 1961

Annual General Meeting and Films. In the Civil Engineering Department, David Keir Building, Queen's University, Belfast, at 6.30 p.m. Tea will be served from 5.45 p.m. to 6.30 p.m.

Hon. Secretary : L. Clements, A.M.I.Struct.E., A.M.I.C.E., A.M.I.Mun.E., 3, Kingswood Park, Cherryvalley, Belfast, 5.

SCOTTISH BRANCH

The following meeting has been arranged :—

Tuesday, 18th April, 1961

Annual General Meeting. At the Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, commencing at 7 p.m.

Hon. Secretary : W. Shearer Smith, M.I.Struct.E., A.M.I.C.E., c/o The Royal College of Science and Technology, George Street, Glasgow, C.1.

SOUTHERN BRANCH

The following meetings have been arranged :—

Wednesday, 8th April, 1961

Visit to Winfrith Atomic Energy Establishment.

Friday, 21st April, 1961

Visit to new buildings for University under construction.

Annual General Meeting, followed by films.

Unless otherwise stated, all meetings will be held at the Engineering Lecture Theatre, University of Southampton, at 6.30 p.m., preceded by tea at 6 p.m.

Hon. Secretary : A. P. K. Tate, B.Sc., A.M.I.Struct.E., Department of Civil Engineering, The University, Southampton.

SOUTH-WESTERN COUNTIES SECTION

The following meeting has been arranged :—

Friday, 21st April, 1961

Annual General Meeting of the Section, at the Duke of Cornwall Hotel, Plymouth, at 6 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary : C. J. Woodrow, J.P., " Elstow," Hartley Park Villas, Mannamead, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

Hon. Secretary : W. D. Hollyman, A.M.I.Struct.E., 41, Greenfield Avenue, Dinas Powis, Glamorgan.

WESTERN COUNTIES BRANCH

The following meeting has been arranged :—

Friday, 7th April, 1961

Annual General Meeting, 6 p.m., followed at 6.45 p.m. by a paper, " Mechanical and Electrical Problems associated with Multi-Storey Construction " by Mr. G. K. Medlock, J.P., A.M.I.Mech.E., and a film, " The Pirelli Building in Milan."

In the Small Lecture Theatre of the University Engineering Laboratories, University Walk, Bristol, 8, at 6 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary : A. C. Hughes, M.Eng., M.I.Struct.E., A.M.I.C.E., 21, Great Brockeridge, Bristol, 9.

YORKSHIRE BRANCH

Owing to unforeseen circumstances, it has been necessary to change the date of the Annual General Meeting, which was announced in the Sessional Programme as the 19th April. The meeting will now be held on Wednesday, 26th April, 1961, at the Metropole Hotel, King Street, Leeds, at 6.30 p.m. (preceded by buffet tea at 6.15 p.m.) and will be followed by a paper, " Timber as a Structural Material," by Mr. I. H. Paxton, M.A.(Cantab), A.M.I.Struct.E.

Hon. Secretary : W. B. Stock, A.M.I.Struct.E., 34, Hobart Road, Dewsbury, Yorks.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary : E. B. Kretzchmar, A.M.I.Struct.E., P.O. Box 3306, Johannesburg, South Africa.

Natal Section Hon. Secretary : J. C. Panton, A.M.I.Struct.E., A.M.I.C.E., c/o Dorman Long (Africa) Ltd., P.O. Box 932, Durban.

Cape Section Hon. Secretary : R. F. Norris, A.M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

EAST AFRICA SECTION

Hon. Secretary : K. C. Davey, A.M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya.

SINGAPORE AND FEDERATION OF MALAYA SECTION

Hon. Secretary : J. R. M. MacIntyre, A.M.I.Struct.E., c/o Redpath Brown & Co. Ltd., P.O. Box 648, Singapore.

NIGERIAN SECTION

Hon. Secretary : A. Brimer, A.M.I.Struct.E., Brimer, Andrews & Nachshen, Private Mail Bag 2295, Lagos, Nigeria.

AUCKLAND (NEW ZEALAND) SECTION

Hon. Secretary : A. Donald, B.Sc.(Hons.), A.M.I.Struct.E., 122, Matipo Road, Te Atatu, Auckland, New Zealand.